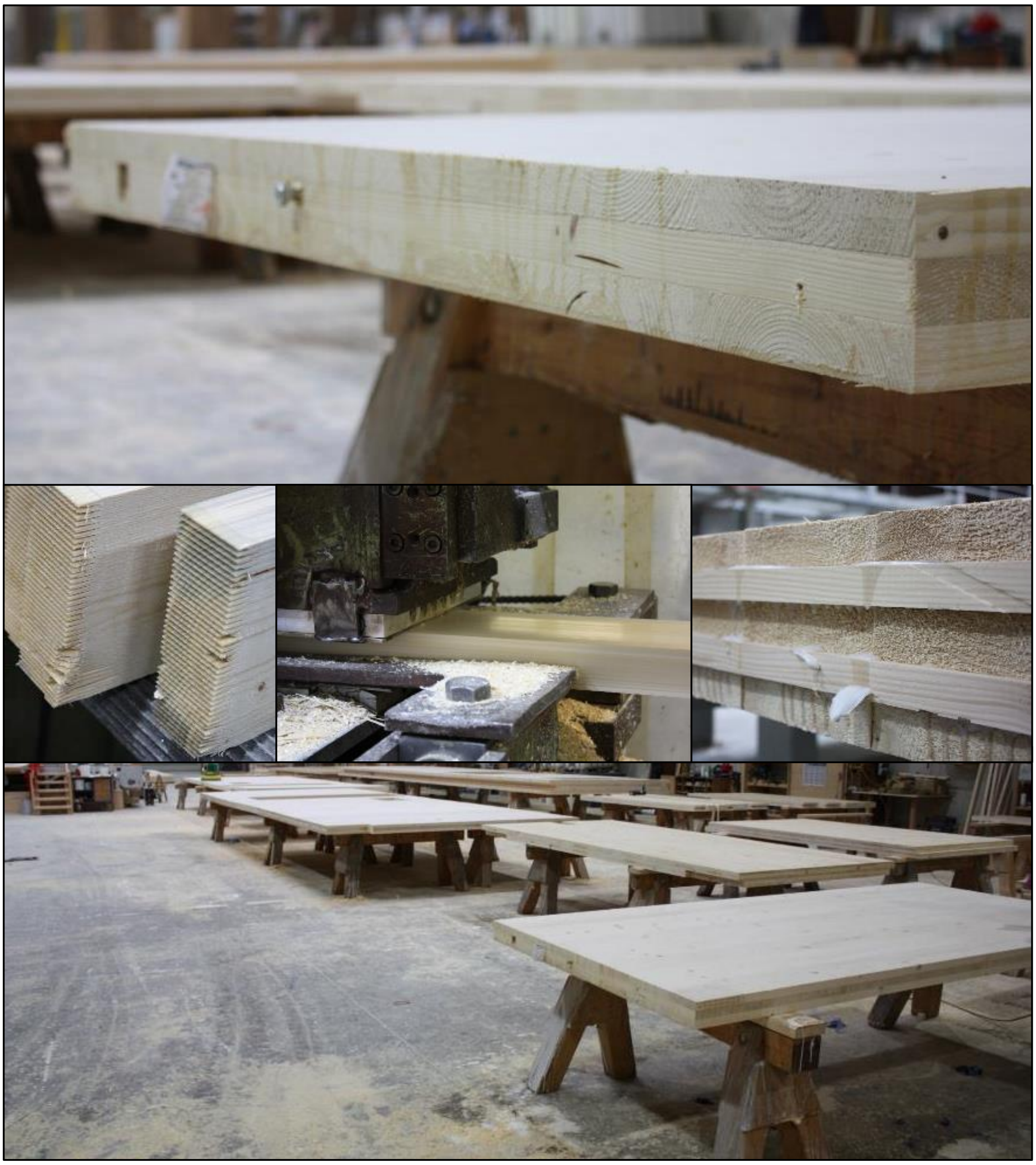
HOLLOW CORE CROSS-LAMINATED TIMBER

OPTIMIZED FOR A MORE EFFICIENT USE OF MATERIAL

By

Bart van Aken



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Thesis

Master of Science in Civil Engineering

By

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Faculty of Civil Engineering and Geosciences

Delft University of Technology

26 June 2017

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ACKNOWLEDGEMENTS

I would like to acknowledge all the people who made a contribution and helped me write this thesis.

First, I would like to express my appreciation to the members of my graduation committee: Prof.dr.ir. J.W.G. van de Kuilen, dr.ir. G.J.P. Ravenshorst, drs. W.F. Gard, dr.ir. P.C.J. Hoogenboom and ir. L.J.M. Houben. With special thanks to Geert Ravenshorst. Thank you for the long discussions, your guidance and patience during my work.

I would also like to thank Dipl.-Ing. M. Derix and the employees of the company W. u. J. Derix GmbH & Co for welcoming me to visit, giving me a tour of the manufacturing process, guiding me and providing me with information. Thank you for making me feel welcome.

Finally, I thank all the people not mentioned above. Thank you for contributing by sharing your insights, opinions and interest.

Delft, June 2017

Bart van Aken

ABSTRACT

Wood is an important building material that has been used for thousands of years throughout the world. It is the most widely used building material and due to its characteristics suitable for a wide range of applications. Still, for the last century steel and concrete have been the materials of choice for large buildings. Part of the reason for this is the lack of knowledge in the industry regarding timber construction. Now that the environmental problems such as climate change get more and more attention, there is an opportunity for timber as a building material and the manufacturers of timber products to increase its popularity.

Cross-Laminated Timber, or CLT, is a relatively new product. First introduced in the early 1990's in Austria and Germany, now gaining popularity in residential and non-residential applications in several countries around the world. The panels normally have 3 or more layers of side-by-side placed boards, that are stacked crosswise at a 90 degree angle. The boards are glued on their wide faces and may or may not be glued together on their narrow faces.

The Derix Group, a company with a vast experience in the timber industry, specializes in laminated timber constructions and are interested in developing a new panel configuration. The idea is that by relocating boards from the middle layer of a CLT element, to a more favourable position, material usage can be optimized.

A problem definition is formulated and used to derive the research questions that form the direction of the thesis. The research questions are related to the possible advantages and the consequences of the Hollow Core Cross-Laminated Timber (HCCLT) configuration.

The first thing that is done in order to investigate the possibilities, is research into the manufacturing process and the structural design of CLT. As a relatively flexible building material with a low dead weight, the design of CLT is often determined by serviceability criteria, such as deformation and vibration. However, the rolling shear properties of the cross-layers can control the design, it influences the effective bending stiffness and stress distribution. It also causes a larger deformation due to shear than for other wood based products. Since there is not one specific design method, the most common methods are researched and described.

- The composite theory
- The mechanically jointed beams theory
- The shear analogy

To determine how the existing methods perform and to investigate the structural behaviour of a HCCLT element compared to a CLT element, the methods are used to calculate different configurations in order to compare the results.

The knowledge obtained during the research and calculations is then used to model a HCCLT element. Different models are made in order to investigate the behaviour during manufacturing and for when the element is in use.

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1 INTRODUCTION

1.1 WOOD AS A BUILDING MATERIAL

Wood is an important building material that has been used for thousands of years throughout the world. It is the most widely used building material and due to its characteristics suitable for a wide range of applications. Resulting in a rich and impressive history of buildings and constructions that have proven themselves, over long periods of time, as being safe and durable structures.

As a natural resource, wood is widely available and with proper management it can provide an endless supply of timber and other wood-based products.

Made out of cellulose, hemicellulose, lignin and extractives, wood is highly anisotropic. 90 to 95% of all cells are aligned parallel to the tree trunk. Shrinkage and swelling of wood in the transverse direction is up to 20 times larger compared to that in the longitudinal direction. The stiffness in the longitudinal direction is normally 20 to 40 times that of the transverse direction. Accounting for "defects" knots, fibre angle etc., which are present in structural timber, will significantly reduce the material properties present in clear wood.

There is a high variability in the strength and stiffness properties of timber. Produced by nature, differences in climate and forestry practices will influence the properties. In order to efficiently work with the material the timber needs to be graded and classified into different qualities. "Defects" can be removed from the material to produce a stronger product.

The maximum size of timber is no longer determined by the size of the trees. Laminated timber has been used as a building material since the 19th century. It is an industrially manufactured product that has a higher stiffness than solid timber and its used for load-bearing structures. Normally made of spruce, it consists of three or more layers glued together with nearly no limitations regarding the dimensions. [1] [2] [3]

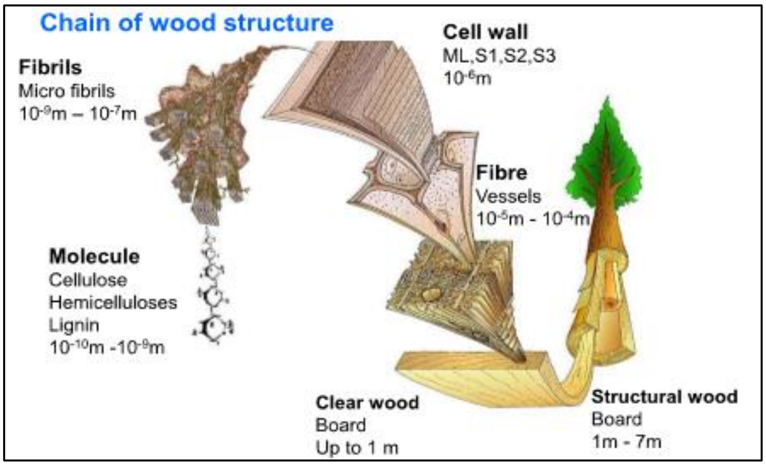


Figure 1.1: Chain of wood structure
Source: University of Canterbury, 1996, Mark Harrington

1.1.1 The structure of wood

Wood can be split into two different categories, hardwoods and softwoods. Both categories contain multiple species with different characteristics. Visual observation does not only show differences between the hardwoods, softwoods and species. Differences are also observed within species, the arrangement of pores, sapwood and heartwood, early- and latewood, the presence of reaction wood and "defects".

A relationship between the width of growth rings and the density can be found for most softwoods and for some ring-porous hardwoods. For softwoods the ratio of the density between latewood and earlywood can be as high as 3:1. The latewood is produced with a relatively constant thickness and most of the variation is caused by the width of the earlywood. This results in an decrease in density for an increase in growth ring width. Caution should be taken when including such relationships as grading parameter in visual grading, the density depends on soil type, climate conditions, etc.

For ring-porous hardwoods the relationship between growth ring width and density is reversed. The high concentration of open vessels that are produced during spring, grow with a relatively constant width. The variation of growth ring width is mainly caused by the thickness of the high density latewood. Which results in an increase of density for an increase in growth ring width. This relationship is only observed in certain ring-porous hardwoods and not in diffuse-porous hardwoods.

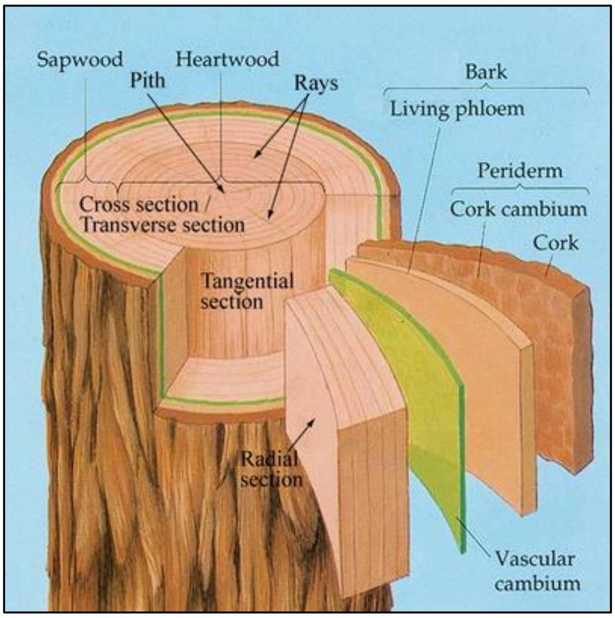


Figure 1.2: A cut-through of a tree trunk
Source: [4]

The young sapwood is located at the outer part of the tree trunk. It's function is to conduct and store the sap from the roots to the crown of the tree. The inner part of the tree trunk is known as the heartwood, which is physiologically inactive. Due to organic extractives the heartwood is better resistant to decay and wood boring insects. The heartwood normally has a lower moisture content, darker colour and a reduction in permeability. Due to the permeability the sapwood is often preferred for wood preservation.

The first 5 to 20 growth rings are called the juvenile wood, often completely located in the heartwood, the juvenile wood has different properties than that of mature wood. The mechanical properties, such as strength and stiffness, are lower and longitudinal shrinkage is normally greater. Forest management plays an important role with respect to juvenile wood, fast grown and short rotation plantation trees can increase the problem attached to juvenile wood for the timber industry.

During growth the tree may be subjected to external forces, such as wind. In order to react, softwoods develop compression wood in areas of high compression and hardwoods develop tension wood in areas of high tensile stress. Compression wood can cause problems with respect to timber engineering. High deformation upon drying and an increased change of brittle failure are the reasons that quality grades have a maximum amount of compression wood that may be present. The presence of tension wood is not as important.

Knots and grain deviation impair the properties and limit the possible use of timber. During growth these flaws occur in the wood and the degree of influence is determined by type, size and location.

Grain deviation in the form of growing a helix around the trunk of a tree can occur in certain species and is most pronounced in younger trees. This "defect" will influence the properties of the timber sawn from such a tree and the timber can turn out to be unsuitable for use. Grain deviations of 1 in 10 and 1 in 5 are included in visual strength grading for respectively high quality timber and low quality timber.

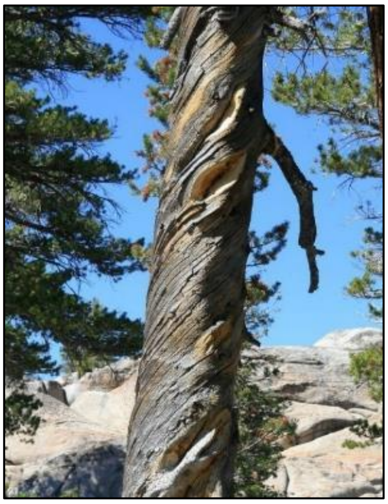


Figure 1.3: Extremely spiral grain
Source: [5]

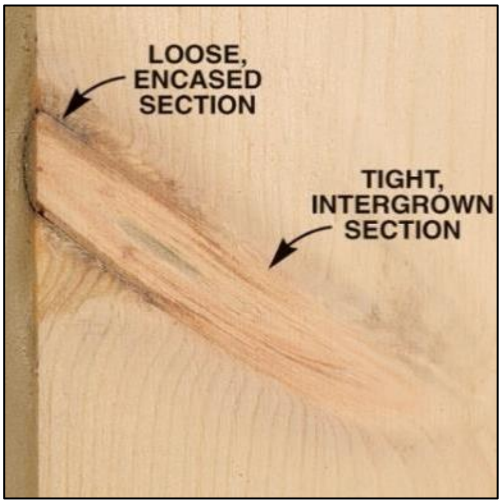


Figure 1.4: Spike knot
Source: [6]

A knot is part of a branch that is embedded in the wood, which disturb the structure of the wood. When successive growth rings form continuously over the branch an intergrown knot is formed, also called a tight knot because it's tightly bound to the wood around it. If the knot becomes encased, by either dying or breaking off. It often has bark surrounding it, separating it from the trunk and becomes an encased knot, also called a loose knot because the bark prevents the knot from tightly binding to the wood around it. Knots form the single most important "defect". [4] [5] [6]

1.1.2 Wood characteristics

The most important physical characteristic of wood is its density. The density is positively correlated to most mechanical properties and the load carrying capacity of joints. Depending on the moisture content, most frequently used are the density of dry wood and that of wood with a 12% moisture content. Values given in the EC5 correspond to a temperature of 20 °C and a relative humidity of 65%.

Wood is a hygroscopic material, which means that it exchanges moisture with its surroundings. Since climate conditions and the environment are constantly changing so does the moisture content in the wood. This can be of great influence on almost all engineering properties of wood. In certain applications the moisture content can reach an equilibrium, but this may need weeks or even months.

When moisture is absorbed into the wood it swells up and when moisture is removed it shrinks. Shrinkage and swelling does not occur uniformly because of the structure of wood. The deflections due to dimensional movement should be minimized. A good start is to use timber with a moisture content that corresponds to the temperature and relative humidity of the environment in which it will be applied.

Distortion can occur during drying. Growth rings have the tendency to straighten out by the difference in tangential and radial shrinkage. Radial cracks form in order to release internal stresses. Different movements in tangential and radial direction are the reason for cup. And lengthwise distortions known as bow, spring and twist may appear due to the presence of compression wood, juvenile wood or knots. Strength grading rules often include maximum values for these distortions.

Further, the mechanical properties of wood are influenced by the moisture content and the duration of loading. Service classes and load duration classes consider the conditions and are used to determine the modification factor. [1]

1.1.3 Building examples

One of the oldest wooden buildings in the world is located in Nara, Japan. The five-storey pagoda of the Horyu-ji Temple was built 1400 years ago and despite the high seismic activity and wet environment still stands today.

Figure 1.5: Horyu-ji Temple, with the pagoda on the right
Source: Yvan Pointurier

The wooden church of Urnes was built in the 12th century and shows the traditional Scandinavian wooden architecture. The stave churches are among the most elaborate and technologically advanced wooden structures of that time in North-Western Europe. The Urnes Stave Church is listed as a World Heritage Site by UNESCO.

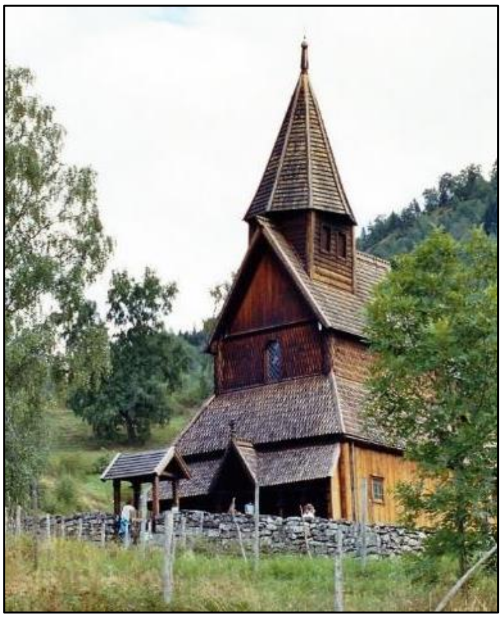


Figure 1.6: Urnes Stave Church
Source: UNESCO, Vesna Vujicic-Lugassy

Figure 1.7: Stadthaus
Source: Mapolis architecture, Will Pryce

The nine-storey high-rise building called Stadthaus, located in London, United Kingdom, is a good modern example. When construction was finished in 2009, it was by far the tallest residential building entirely made of wood, which include: walls, floors, roofs, elevator shafts and stairwells. To achieve this, they made use of the relatively new product Cross-Laminated Timber.

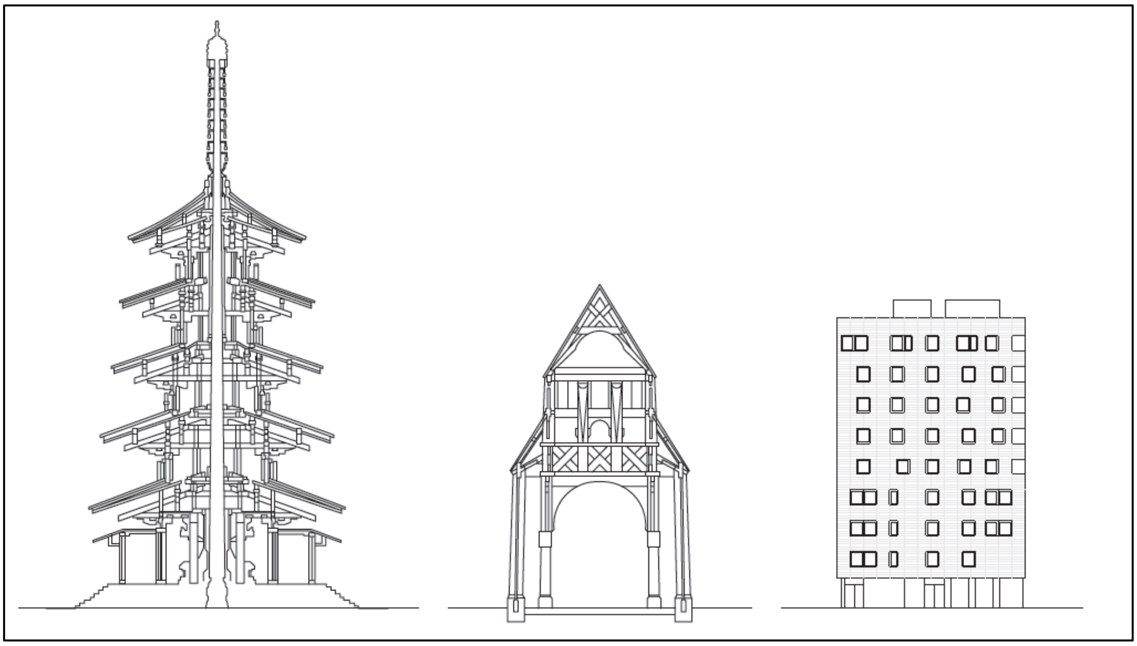


Figure 1.8: From left to right, Horyu-ji Temple Pagoda, Urnes Stave Church, Stadthaus
Source: [7]

1.2 TIMBER VS CONCRETE & STEEL

For the last century steel and concrete have been the materials of choice for large buildings. Part of the reason for this is the lack of knowledge in the industry regarding timber construction. Now that the environmental problems such as climate change get more and more attention, there is an opportunity for timber as a building material and the manufacturers of timber products to increase its popularity.

The material properties and life-cycle balance of timber are outstanding and as a nearly endless renewable natural resource it can provide an ecologically sound alternative to steel and concrete.

Physically, wood is a strong and stiff material, but compared to steel it is also light and flexible. Advantages of wood over steel can be found in the costs, its high strength over density ratio and the low energy output.

Timber elements that make use of a more solid wood configurations can be best compared to heavy construction systems. Concrete is often used for large floor and wall elements and has evolved by introducing steel-reinforcement, higher strength classes and hollow core slabs.

Hollow core slabs are precast concrete elements that are generally used for flooring. The elements are pre-stressed and have cores running through them in longitudinal direction. The main purpose of these cores is to reduce the amount of material and weight, while maintaining strength. Compared to solid concrete floors the precast elements can save up to 50% in material. The width is typically 1200 mm and the elements can be produced with lengths up to 18 m. Common lengths, for example used in office buildings, are 7200 mm and 14400 mm. [8]

Figure 1.9: Concrete hollow core slab
Source: [8]

Now, that there is more attention and awareness concerning the environmental impact of the building industry. There is a growing interest in wood as a building material. The reason for this is that few building materials possess the environmental benefits that wood has. As a nearly endless renewable natural resource wood can make a difference.

1.3 CROSS-LAMINATED TIMBER

Cross-Laminated Timber, or CLT, is a relatively new product. First introduced in the early 1990's in Austria and Germany, now gaining popularity in residential and non-residential applications in several countries around the world.

CLT is a engineered wood building system with potential applications in high-rise structures. Known as massive (or "mass") timber the heavy construction system works on the same load bearing principals as large floor and wall elements made of concrete or brick. As seen in the example of the Stadthaus building, there is a wide area of applications. Solid panels can be used for the walls, floors, roofs and even elevator shafts and stairwells.

Panels are made from several layers of side-by-side placed boards, stacked crosswise at a 90 degree angle. The boards are glued on their wide faces and may or may not be glued together on their narrow faces. The panels normally consist of 3 or more layers, with a symmetrical build-up around the middle layer.

Panel sizes vary by manufacturer and it is most likely that transportation will limit sizing. Panels are made with dimensions up to 18 m in length by 3,5 m in width. Thickness is limited to 500 mm.

As a result of the cross-laminating the panels possess an improved dimensional stability, strength and rigidity. This allows for the fabrication of large wall and floor elements that can be used for bracing and load bearing implementations. [9] [10] [11]

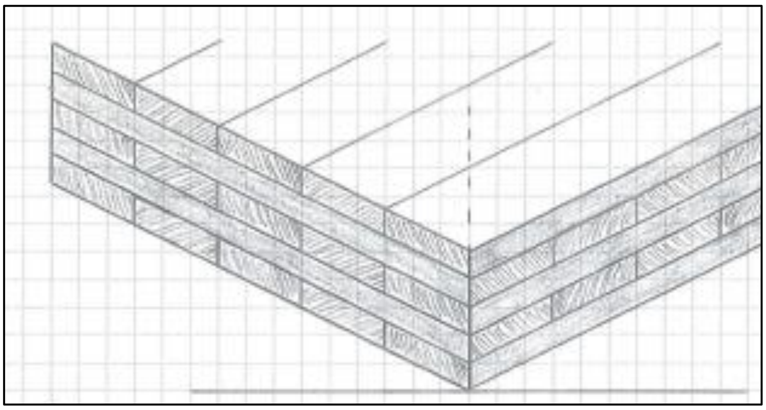


Figure.1.10: CLT panel configuration

Figure 1.11: FORTÉ
Source: [12]

Aesthetics, cost effectiveness and environmental performance are good reasons to use wood as a building material. However, CLT introduces several additional benefits of its own.

Prefabrication for specific applications, with pre-cut openings for doors, windows, service channels etc. contributes to a quick and efficient installation on site. The flexibility in design and the possibility to make modifications on site with the use of simple tools further improve on this.

CLT has an excelled fire resistance and unlike steel it remains structurally stable when subjected to high temperatures. As a combustible material, wood does ignite at high temperatures. However, compared to other combustible materials the fire will spread slowly and the surface will burn at a steady rate. The burned area of the wood will form a layer of char and loses its strength, after which it will become an insulating layer protecting the core of the panel. Since wood is a poor conductor of heat the core of the panel will be nearly unaffected and keep its properties. As a result, the loss in capacity is only due to the reduced cross-section.

Seismic testing on multi-storey CLT buildings shows that the panels perform exceptionally well. In Japan a seven-storey CLT building showed no residual deformation after severe earthquake simulation. This is thanks to the dimensional stability and rigidity of CLT combined with its ductile behaviour and energy dissipation.

Finally, the precise manufacturing process gives CLT buildings additional benefits in terms of thermal and acoustic behaviour, by preventing air leakage within the building envelope. [12]

1.4 RESEARCH SPECIFICATION

In this chapter the research is specified. The problem definition is described and the research questions are defined, these will form the basis for the research to be carried out for the thesis.

The Derix Group, a company with a vast experience in the timber industry, specializes in laminated timber constructions and are interested in developing a new panel configuration. The idea is that by relocating part of the cross area to a more favourable position, material usage can be decreased or the strength can be increased.

A configuration somewhat similar to combining CLT-panels and Glued laminated beams is proposed. Figure 1.12 shows the proposed configuration with the idea of pressing the element as a whole. See also appendix: A - Abmessungen und aufbau von x-lam-stegplatten.

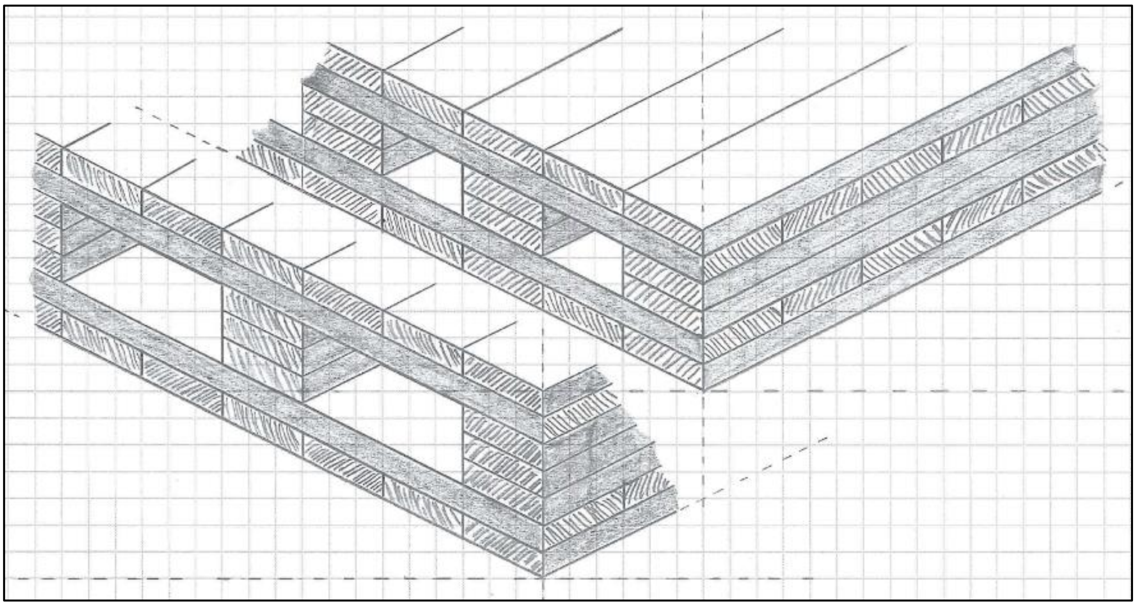


Figure 1.12: Sketch of hollow core cross-laminated timber, two and three layer

1.4.1 Problem definition

Cross-Laminated Timber is a relatively new product which has not been optimized for the wide area of applications it is being used for. In order to reach large spans needed for (industrial) halls and other large structures the CLT elements need to become very thick, which results in an large increase in net cross area. Hollow Core Cross-Laminated Timber, or HCCLT, can decrease this large net cross section.

Wood is an important building material and there is an increase in demand that will grow even further in the coming years.

CLT

Cross Laminated Timber

or

Consumes Lots of Timber [13]

1.4.2 Research questions

The following main and sub questions are derived from the problem definition and will be used to form the direction of the thesis.

What advantages can be realized with Hollow Core Cross-Laminated Timber systems?

> *What are the consequences of the manufacturing process on the configuration and dimensions of the hollow cores?*
>
> *What effect will the hollow cores have on the structural properties of an element?*
>
> *What effect will different dimensions of the hollow cores have on the economical use of material?*

In order to answer the main and sub-questions, the following items must be addressed:

What is the behaviour of the element during production and in use?
- What are the strength and stiffness properties of the element?
- What possible failure mechanisms occur?

What is the structural behaviour of the element?
- Are the deflection and vibration within reason?
- Can the element provide a comfortable environment?

Is it possible to derive an calculation method for the HCCLT elements?
- How do the existing calculation methods perform?
- Is further development of the methods possible?

2 MANUFACTURING

The production of timber starts in the forests, where forest management plays an important role. Trees are grown and cut down to produce the raw material needed for manufacturing. After which the timber needs to dry before it can be processed.

2.1 THE DERIX GROUP

Figure 2.1: Office & factory of the Derix Group, Niederkrüchten, Germany

The Derix Group specializes in the construction of laminated timber. In cooperation with their partners and with over 80 years of experiences in the timber industry they are able to realize the most outstanding projects. Supplying products to Germany, the rest of Europe and abroad.

The CLT elements for the new office building of Poppensieker & Derix GmbH & Co. KG. were prefabricated in the factory in Niederkrüchten, Germany. The building is listed as one of their reference projects. Thanks to the prefabrication the construction time of the two storey building with a floor area of 638 m^2 was minimal. [14]

Figure 2.2: Administration building Poppensieker & Derix, Westerkappeln, Germany
Source: W.u.J. Derix GmbH & Co

One of the most important environmental topics these days is climate change. Since timber is a nearly endless renewable natural resource, the use of timber as a building material can be seen as an active and substantial contribution to protecting the climate. The timber that is used comes from sustainable forestry in Europe, the number of trees that are currently growing is much larger than the number of trees that are cut down for production purposes, there is an annually increase in population. The Derix Group finds the use of timber both responsible and economic for manufacturers as well as constructors.

> Timber is not just a building material like any other building material: Its material properties and its life-cycle balance are outstanding. Timber as a natural construction material is ecologically sound and economically very cost-effective. This is the reason why the use of timber can be called visionary. Using timber helps us to safeguard the future for ourselves and coming generations. As a leading manufacturer in the timber industry it is our aim to contribute to this common purpose. [15]

2.2 MANUFACTURING PROCESS

In this chapter the manufacturing process of Cross-Laminated Timber is described. Starting right after harvesting, with drying of the wood and ending with finishing and transport.

2.2.1 Kiln drying of timber

Figure 2.3: Timber Drying Kiln
Source: Vortex Engineering

Wood is a hygroscopic material, which means that it balances its moisture content with the surrounding atmosphere. When the shrinkage of wood occurs to rapidly, due to drying, internal stresses are introduced. To prevent the timber from cracking by internal stresses and to maximize the bond-strength and stability, the timber is dried in a kiln. A kiln is a closed surrounding in which the temperature, air circulation and relative humidity can be controlled. The timber is dried to a moisture content of 10 to 12 percent.

Compared to air drying it has the advantage of being a much faster method and to obtain the low moisture content is never a problem. However, if the temperature is too high problems can occur which may lead to a reduction of strength. [16]

2.2.2 Storage and grading

Figure 2.4: Timber storage, The Derix Group, Niederkrüchten, Germany

The timber is kiln dried before it arrives at the factory, where it gets stored before being processed. There are multiple storage areas to provide space for the large amount of raw timber.

Figure 2.5: Timber grading, The Derix Group, Niederkrüchten, Germany

Timber gets assigned to a particular strength class, this can be done either by machine strength grading or visual grading. The grading is based on measured parameters, such as the modulus of elasticity, and the strength of the timber. For visual grading the knot size and its position are important.

Grading of the timber is done on the basis of DIN 4074. The density properties get mechanically determined and the timber is sorted by strength and visual appearance. Because of its nature, the mechanical properties of timber vary and since this determines the loadbearing and deformation capacity it is very important to grade the timber accurately.

Undesirable "defects" like large knots are marked by hand and removed from the boards. The different length boards continue to the finger jointing machine.

2.2.3 Finger jointing

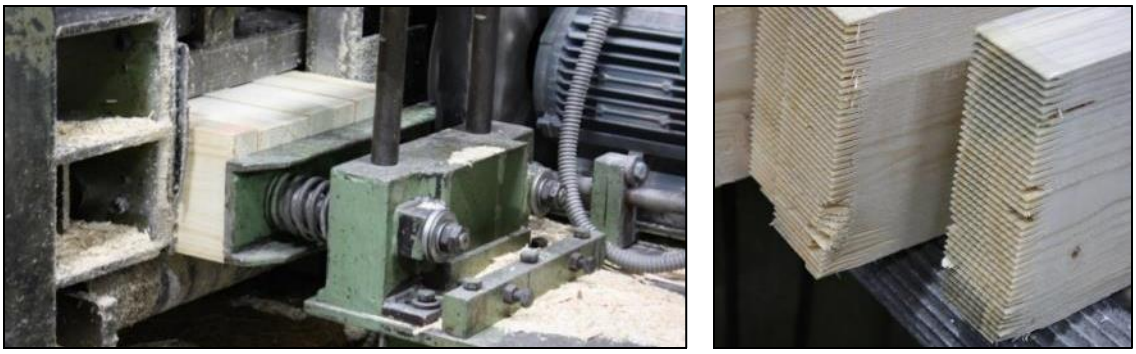


Figure 2.6: Finger jointing, The Derix Group, Niederkrüchten, Germany

After grading, the boards go through the finger jointing machine. The first step consist of machine cutting the profile into the boards, as shown in Figure 2.6. This is done on both sides of the boards so they can be glued and pressed together, making a continuous member.

2.2.4 Planing, cutting and separating

Figure 2.7: Planing and cutting of the "endless" board, The Derix Group, Niederkrüchten, Germany

After realizing a continuous member or an “endless” board it gets planed to the required thickness and then cut to the desired length, see Figure 2.7. Planing just prior to the application of adhesive insures a strong and durable bond, by “refreshing” the wood surface to reduce oxidation. After which the production line separates the boards that are used for the production of Glulam and the production of CLT.

2.2.5 Gluing, stacking and pressing

Figure 2.8: Boards for the cross layers, The Derix Group, Niederkrüchten, Germany

Because of the high accuracy / low tolerances for the length of the boards, needed in order to get a good result after pressing. The boards are recut to size, for the transverse direction as well as the longitudinal direction. After which the different layers will be laid out and transported to the press, see Figure 2.8 and Figure 2.9.

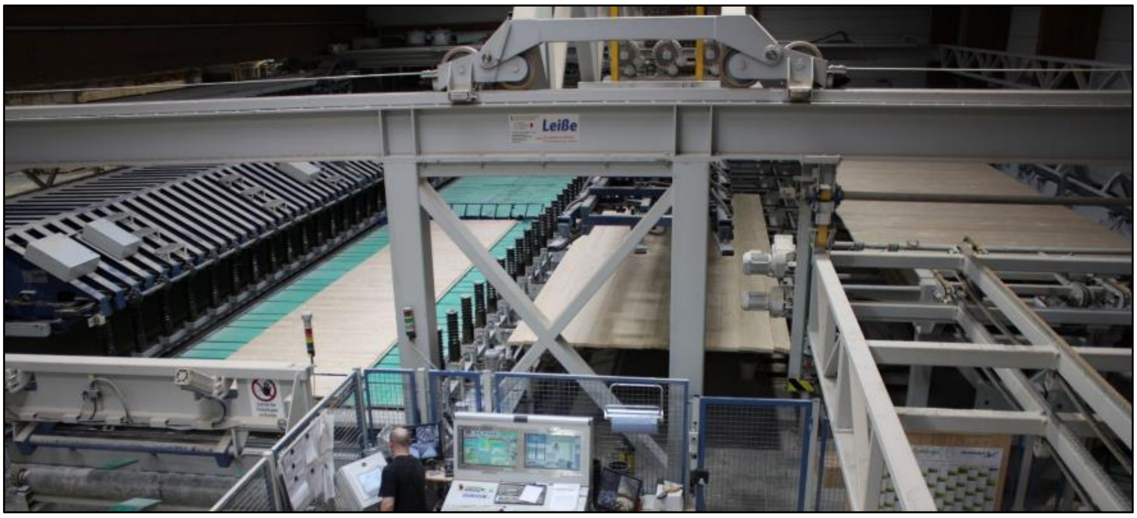


Figure 2.9: Build-up of a CLT-element, The Derix Group, Niederkrüchten, Germany

Proper bond development and CLT quality depend greatly on the pressing process. The two main types of presses used to produce CLT are vacuum and hydraulic. The maximum theoretical pressure of a vacuum press is 0.1 MPa. This low pressure may not be sufficient when warping, surface irregularities, etc. are present. Shrinkage reliefs in the form of longitudinal cuts, through partial thickness of the boards, can reduce stress. These shrinkage reliefs can however influence the performance of the CLT and should be tested as part of the product qualification. Hydraulic presses have a much higher clamping pressure.

Assembly time and pressing time depend on temperature and humidity. Adhesive manufactures provide minimum requirements, such as minimum temperatures, because the adhesive may take longer to cure at low temperatures. Product information can be found in appendix: B - Product information grippro^{tm} plus.

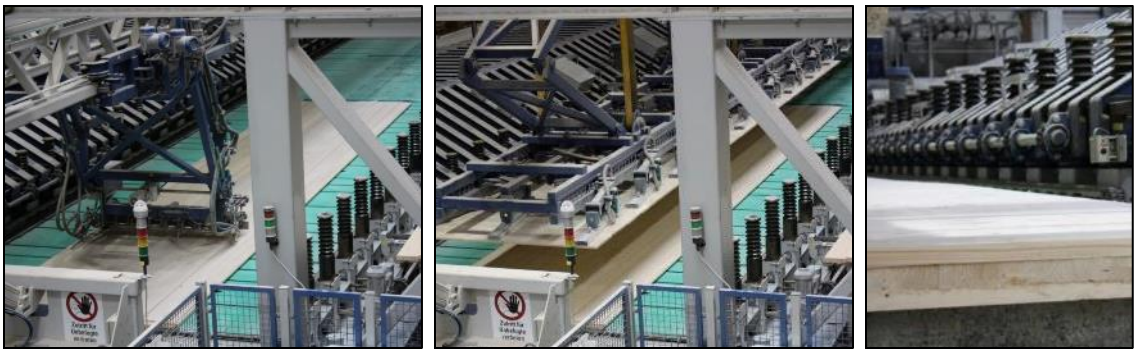


Figure 2.10: Gluing, stacking and pressing of a CLT-element, The Derix Group, Niederkrüchten, Germany

A layer of boards, depending on the purpose of the element, this can be a longitudinal layer in case of a floor or roof element or a transverse layer for wall elements, is placed on the bottom surface of the press. Adhesive is applied and a next layer of boards can be placed on top, this will continue until the required composition is reached. In certain cases, two adjacent layers can be aligned in the same direction in order to meet specifications. When this is finished the press closes and pressure is applied.

The press can be either hydraulic or vacuum. Hydraulic presses are more common and is the type of press used at Derix. Elements can be produced up to 400 mm thick with a length and width of respectively 17,8 and 3,5 m.

There is no glue applied on the narrow faces of the boards at Derix only on the wide faces, which some calculation methods will account for. The pressure ranges between 0.5 to 0.8 MPa, depending on the square footage under the press. Pressing time depends on equipment and adhesive, the pressing time at Derix is around 60 min.

2.2.6 Inspection and repairs

The CLT element is pressed and ready for inspection and repairs. Knots and other “defects” get removed and replaced by a new piece of timber which is then sanded down to produce a smooth and well-made finish, see Figure 2.11.

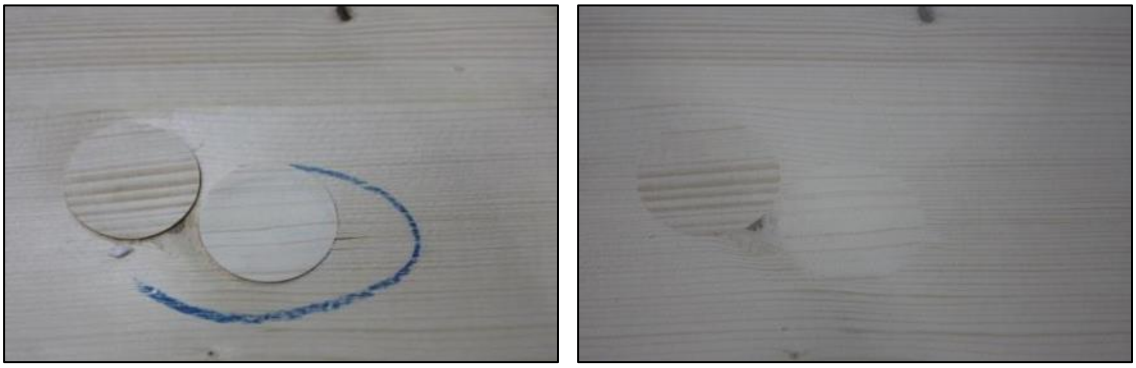


Figure 2.11: Necessary repairs are carried out, The Derix Group, Niederkrüchten, Germany

In Figure 2.12 it can be seen that the layers do not align perfectly, this is needed for the pressing process. Although the sides of the boards are not glued, so there is no edge bond present. Boards placed side-by-side are pressed towards each other to minimize the gap between them. The elements will be further processed by the CNC-machine, where the sides get their final finish.

Figure 2.12: Pressed CLT-element, The Derix Group, Niederkrüchten, Germany

2.2.7 CNC-machining

Figure 2.13: CLT-elements and machine for 180 degrees turning, The Derix Group, Niederkrüchten, Germany

After producing the large CLT elements, the final products are realized with the help of CNC-machines. These machines are used for machining (sawing, drilling, milling, etc.) the elements, window and door openings are made as well as room for wiring and sockets. Further preparation, on for example connections, can be done to simplify the erection on site.

In order to be able to work on both sides of the elements there is a separate machine for turning the workpiece 180 degrees, which can be seen in the upper left corner of Figure 2.13. This makes for a quick and flexible system.

Figure 2.14: CNC-machining and final product, The Derix Group, Niederkrüchten, Germany

2.2.8 Finishing

Figure 2.15: Finishing by sanding and applying varnish, The Derix Group, Niederkrüchten, Germany

There are multiple possible surface qualities from which can be chosen. Depending on requirements and preference, the products can be made to the following qualities: non-visible (NSI), visible industrial (ISI) and living-space (WSI).

The layout of the boards, with other words the orientation of the growth rings, is chosen for a better visible quality. And it does not depend on the structural properties.

The varnish applied in the factory is only to protect the product in the erection stage of the build.

Figure 2.16: Finished element, The Derix Group, Niederkrüchten, Germany

2.2.9 Transport

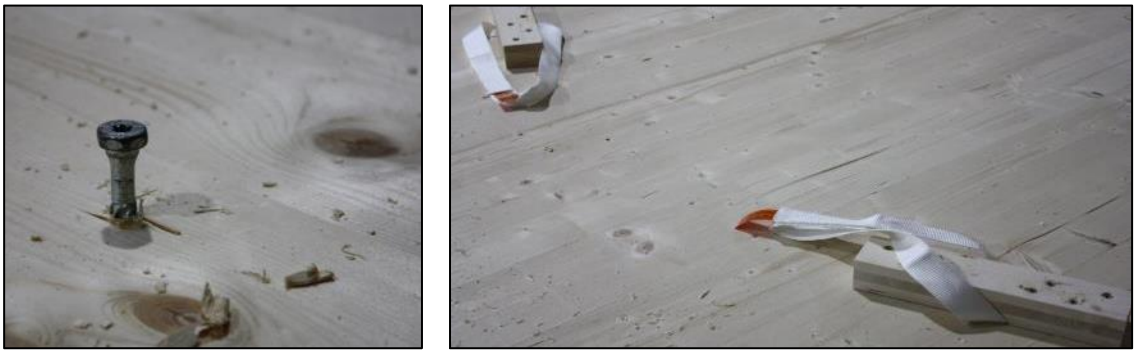


Figure 2.17: Temporary transport measures, The Derix Group, Niederkrüchten, Germany

Various temporary constructions are used depending on size and weight to lift and transport the elements. Because of the low self-weight of timber, transport is easily realized.

However transport to site can be costly and may require specialized services, depending on the size of the elements. The route to the construction site and the construction site itself could have restrictions concerning the size.

Figure 2.18: Transporting the final product, The Derix Group, Niederkrüchten, Germany

3 STRUCTURAL DESIGN OF CLT

The different applications CLT is used for determines the stresses introduced into the elements. Floor elements are introduced by stresses as a result of loading perpendicular to the plane of the panel and wall elements by loading in the plane of the panel. Additionally, the strength and stiffness depend on the direction of the loading, parallel or perpendicular to the grain of the outer layers.

The build-up determines the load bearing performance and the primary direction of the load bearing capacity generally corresponds to the orientation of the outer layers. With the outer layers running parallel to the action of the load it insures that the structural elements are optimally oriented. For wall elements this means that the outer layers run vertically and for floor elements that the outer layers are oriented in the direction of the span.

The strength grades to produce CLT are normally C16 (mostly used for the inner layers) and C24, material properties of the latter can be found in the table below.

Table 3.1: Material properties C24

Material properties C24 Strength and stiffness properties in N/mm^2		
Bending strength	fm;k	24
Tensile strength	ft;0;k	14
	ft;90;k	0,4
Compression strength	fc;0;k	21
	fc;90;k	2,5
Shear strength	fv;k	2,7
	fr;k	1
Modulus of elasticity	E0;mean	11000
	E90;mean	370
Shear modulus	Gmean	690
	Gr;mean	50
Density [kg/m^3]	ρk	350
	ρmean	420

The strength has proven to be increased by the composition of CLT and can be accounted for by introducing one of the following factors:

- Laminating effect
- Size factor
- Strength factor

3.1 DESIGN CRITERIA

CLT is a relatively flexible building material with a low dead weight. The design is often determined by serviceability criteria, such as deformation and vibration. Strength criteria, like bending and shear are mostly not governing.

Every structure has to uphold the requirements of safety and serviceability. While the serviceability requirements normally do not influence the risk to human live, they do influence the comfort in use.

In some cases where the serviceability requirements are no longer met, there could be a solution to reverse the situation. Vibrations that cause human discomfort may be reversible, as is the visual appearance and functionality of a floor. [17]

Limiting values for the design of timber structures are formulated with respect to the deflection and vibration.

3.1.1 Deformation

The total deformation can be divided into the deformation due to permanent and variable loading. Additional factors for creep and time further determine the final deformation.

Reasons for limiting the deformation of elements are related to appearance (visual effects, such as bending of floors) and structural requirements (ensured functionality of building and installations, avoid damage to doors, windows and to guarantee smooth assembly, water tightness, etc.).

Section 2: Basis of design of the Eurocode 5 – Design of timber structures – Part 1-1: General – Common rules and rules for buildings, states that for structures consisting of members, components and connections with the same creep behaviour and under the assumption of a linear relationship between the actions and the corresponding deformations, as a simplification of 2.2.3(3), the final deformation, u_{fin}, may be taken as:

$$u_{fin} = u_{fin,G} + u_{fin,Q1} + u_{fin,Qi} \tag{3.1}$$

where:

$$u_{fin,G} = u_{inst,G}\ (1 + k_{def}) \tag{3.2}$$

$$u_{fin,Q1} = u_{inst,Q1}\ (1 + \Psi_{2,1}\ k_{def}) \tag{3.3}$$

$$u_{fin,Qi} = u_{inst,Qi}\ (\Psi_{0,i} + \Psi_{2,i}\ k_{def}) \tag{3.4}$$

For an office building, $\Psi_2 = 0{,}3$, according to Table NB.2 – A1.1 – Ψ-factors for buildings from the Dutch National Annex to NEN-EN 1990+A1+A1/C2: Eurocode: Basis of structural design.

Table 3.2 – Values of k_{def} for timber and wood-based materials from Eurocode 5: Design of timber structures – Part 1-1: General – Common rules and rules for buildings, includes values for solid timber, glued laminated timber, LVL, plywood, etc.

For service class 1 the values of k_{def} for solid timber and plywood are respectively 0,6 and 0,8. The configuration of CLT with timber spanning in two directions may be best compared to that of plywood, due to the influence of the cross-layers on the creep of the element.

At the University of Applied Sciences in Augsburg the creep behaviour of CLT has been investigated. The results indicate that the creep behaviour of CLT is more pronounced than for glued laminated timber and therefore higher values for k_{def} should be used. [18]

For the calculations in this thesis the value for k_{def} is taken according to the CLT Handbook. The CLT Handbook includes deformation modification factors adjusted to CLT (based on recommendations of Jöbstl and Schickhofer, 2007). For service class 1, $k_{def} = 0{,}9$.

Limiting values for the deformation are taken according to Table 7.2 – Examples of limiting values for deflections of beams from Eurocode 5 – Design of timber structures – Part 1-1: General – Common rules and rules for buildings.

$$w_{inst} \leq l/300 \ to \ l/500 \tag{3.5}$$

$$w_{fin} \leq l/250 \ to \ l/350 \tag{3.6}$$

3.1.2 Vibration

Loading situations that occur with normal use, should not introduce vibrations that have a negative influence on the construction or the comfort of the users.

The Eurocode 5 includes design criteria that apply to a fundamental frequency higher than 8 Hz. Floors with a lower fundamental frequency will have a much larger response to the motion of people and require special investigation. The reason for the low frequency is usually due to large spans.

The requirements in Eurocode 5 are for a rectangular floor with overall dimensions $l \cdot b$, simply supported along all four edges and with timber beams spanning in l direction. An approximation of the fundamental frequency can be calculated with the equation below.

$$f_1 = \frac{\pi}{2 \cdot l^2} \cdot \sqrt{\frac{EI}{m}} \tag{3.7}$$

The mass m in kg/m^2 is taken as the permanent load only, partitions and variable loading are excluded.

Vibration effects can be divided into low- and high-frequency ones. The fundamental frequency of the floor should be at least 8 Hz in order for the floor to be regarded as a high-frequency one.

For residential floors with a fundamental frequency greater than 8 Hz the following requirements should be satisfied:

$$\frac{w}{F} \leq a \; [mm/kN] \tag{3.8}$$

$$v \leq b^{(f_1 \zeta - 1)} \; [m/(Ns^2)] \tag{3.9}$$

w is the maximum instantaneous vertical deflection due to a vertical concentrated static force F applied at any point on the floor.

v is the maximum initial value of the vertical vibration velocity produced by an ideal unit impulse of 1 Ns, applied at the point on the floor giving the maximum response. Components above 40 Hz may be disregarded.

ζ is the modal damping ratio. Unless other values are proven to be more appropriate, a value of 1 % should be assumed.

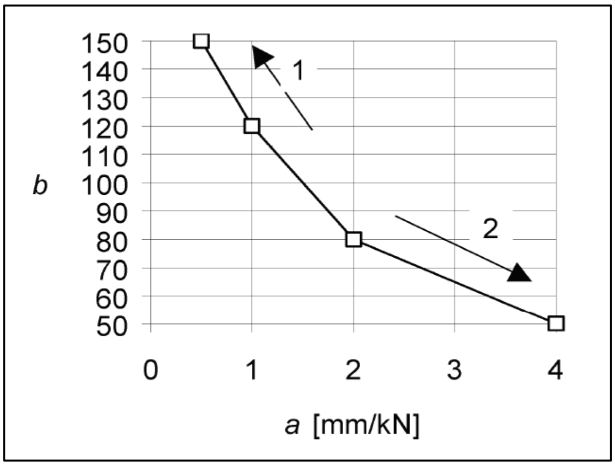


Figure 3.1: Recommended range of and relationship between a and b
Source: [19]

Figure 3.1 shows the recommended range of the relationship between a and b, 1 better performance, 2 poorer performance. Countries are able to state their own values in the National annex.

The Dutch national annex to the Eurocode: Basis of structural design states that if the fundamental frequency of floors used in residential and office buildings stays above 3 Hz the design criteria are not exceeded. And that this criteria does not have to be matched in case the loading situation is more than 5 kN/m^2.

The highest frequency that can be generated by persons walking, amounts to 3 Hz. For this the deflection in the short-term behaviour must not exceed 34 mm. If a load of 5 kN/m^2 is present, it can be assumed that the floor cannot be vibrating brought on by walking persons. [20] [21]

Also important are the vibrations of foundations and surrounding soil. These vibrations need to be limited to an acceptable level. Ideally, people should not feel vibrations in their home or at the office. The perception of vibrations is a function of the acceleration and the frequency. Multiple perception levels can be formulated, ranging from very unpleasant to not noticeable. See Figure 3.2 and Table 3.2 for the perception and the different levels. [22]

Using the equations for the vertical acceleration found in Annex B from Eurocode 5: Design of timber structures – Part 2: Bridges. [23] The perception level found ranges from unpleasant to well noticeable, dependent on the damping ratio. The damping can reach up to 9% for timber floors in combination with certain furniture and finishing. [24]

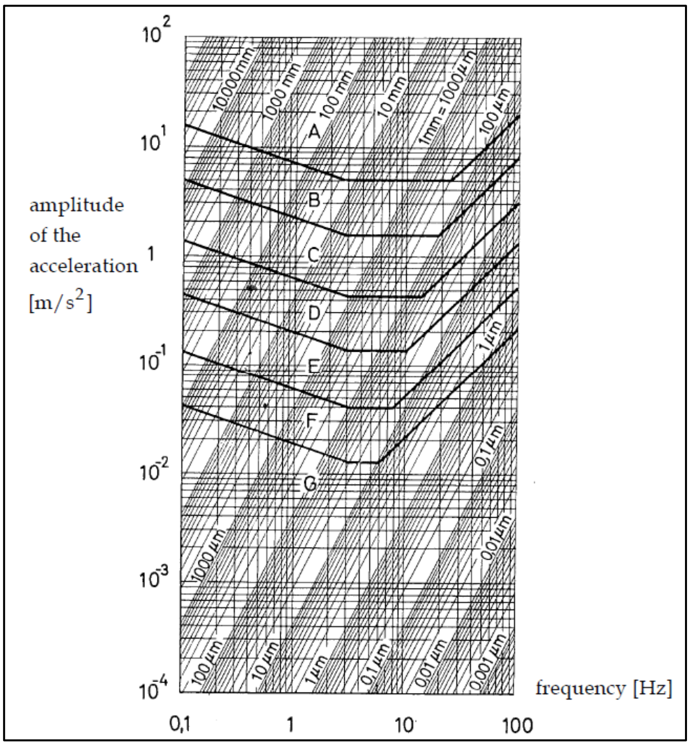


Figure 3.2: Perception of vibration

Table 3.2: Perception levels

	perception	acceptability in buildings	structural effects	examples
A	very unpleasant	not acceptable	danger of collapse	- earthquake
B	unpleasant	not acceptable	local damage	- emergency braking of a car
C	strongly noticeable	hardly acceptable	cracks in masonry	- in a tram or elevator
D	well noticeable	only rough work	small cracks	- start of seasickness
E	noticeable	shortly in rooms	no influence on building	
F	hardly noticeable	acceptable	no influence	
G	not noticeable			

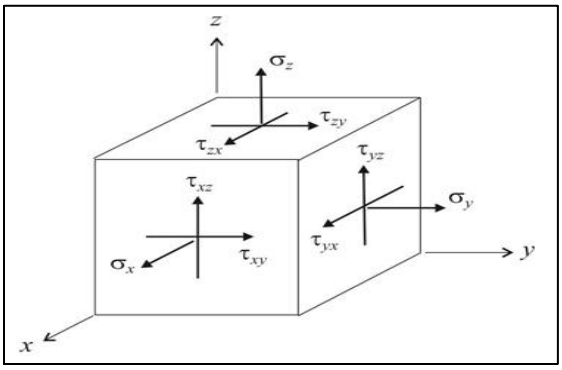
3.2 ROLLING SHEAR

The rolling shear strength and stiffness properties can control the design of CLT. It influences the performance significantly. The rolling shear modulus of the cross-layers affect the effective bending stiffness and stress distribution, and so the load bearing capacity. The reduced rolling shear modulus G_R of the cross-layers in CLT also causes a larger deformation due to shear than for other wood based products. [25] [26]

The rolling shear modulus is influenced by species, density, thickness, moisture content, annual ring orientation, etc. For spruce, common values of the rolling shear modulus are between 40 and 80 N/mm^2.

To determine the rolling shear modulus a ratio of $\frac{G_{R,mean}}{G_{mean}} = 0.1$ can be used. In many publications and calculations the rolling shear modulus is simply taken to be 50 N/mm^2.

In Figure 3.3 six different shear modes are shown.



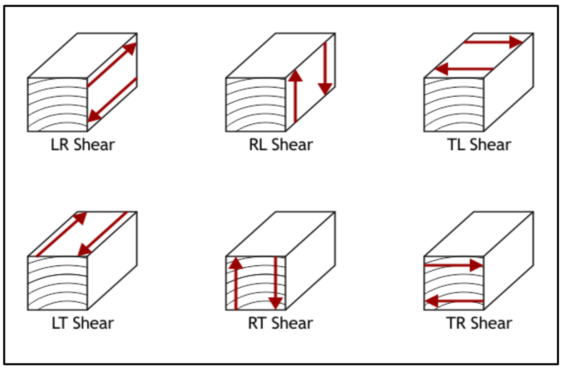


Figure 3.3: Stresses on an element
Source: Lost Art Press

RT and TR shear are called rolling shear. Imagine wood fibres rolling alongside each other. Defined as shear stress leading to shear strains in a plane perpendicular to the grain direction. Significant shear deformations may occur due to the low rolling shear stiffness. Wood can fail readily when subjected to rolling shear.

LR and LT shear are the longitudinal shear modes. Although wood is more resistant to longitudinal shear, it is a common failure mode in an overloaded beam.

RL and TL are the two kinds of transverse shear. Wood rarely fails in transverse shear.

3.3 DESIGN METHODS

There are a variety of different methods being used to design Cross-Laminated Timber structures. The design of CLT is not included in the Eurocode 5: Design of timber structures and at this moment there is not a universally accepted approach for the design.

The Technical Committee CEN/TC 124 "Timber structures", is working on a European Standard, with requirements for CLT, which is currently submitted to the Formal Vote. The document sets out provisions regarding the performance characteristics for structural CLT as a material for the manufacturing of structural elements. [27]

Experimental testing is the most accurate, however changes in build-up, material, or even differences in the manufacturing process make for a non-effective method.

This chapter will describe the three analytical approaches that are most used to date.

3.3.1 Composite theory

For timber engineers this method is well known from the calculation of plywood. For the calculation of plywood the modulus of elasticity perpendicular to the grain is assumed to be zero, which means that the cross layers are not taken into account. However, for the calculation of CLT this assumption can lead to large differences between calculation and test results.

For CLT, the composite theory takes both, the layers loaded parallel to the grain and cross layers loaded perpendicular to the grain into account. The strength and stiffness properties of a single board multiplied by a composition factor k, that accounts for the build-up of the element, are used to determine the strength and stiffness properties of the CLT element.

This calculation method should only be used for high span to depth ratios, since it does not account for shear deformation in bending members. The influence of the shear deformation is significant for smaller span to depth ratios. For loading perpendicular to the plane and parallel to the grain of the outer layers a $ratio \approx 30$ and a $ratio \approx 20$ for loading perpendicular to the plane and perpendicular to the grain of the outer layers. See Figure 3.4 on page 33.

For smaller span to depth ratios the theory of mechanically jointed beams, from Eurocode 5, or the shear analogy method, by Kreuzinger, can be used. The theory of mechanically jointed beams takes the shear deformation into account by the use of a reduction factor γ. The shear analogy method can be used for a more precise calculation, because it uses both different modulus of elasticity and shear modulus.

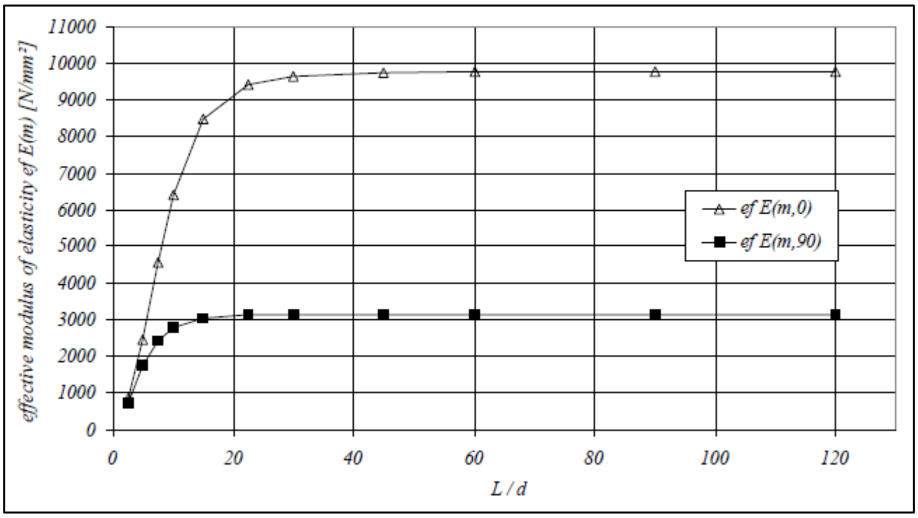
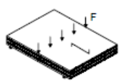
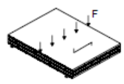
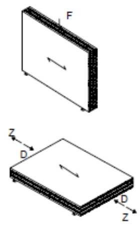
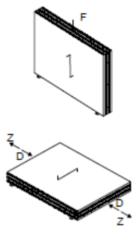


Figure 3.4: Effective modulus of elasticity
Source: [28]

The ratio between the strength or stiffness property of an element and that of a fictitious homogenous cross section with the grain of all layers oriented parallel to the direction of the stress is taken into account by the composition factor k. For the compositions factors see Table 3.3.

Table 3.3: Compositions factors k
Source: [28]

	k_i
	$k_1 = 1 - \left(1 - \frac{E_{90}}{E_0}\right) \cdot \frac{a_{m-2}^3 - a_{m-4}^3 + \ldots \pm a_1^3}{a_m^3}$
	$k_2 = \frac{E_{90}}{E_0} + \left(1 - \frac{E_{90}}{E_0}\right) \cdot \frac{a_{m-2}^3 - a_{m-4}^3 + \ldots \pm a_1^3}{a_m^3}$
	$k_3 = 1 - \left(1 - \frac{E_{90}}{E_0}\right) \cdot \frac{a_{m-2} - a_{m-4} + \ldots \pm a_1}{a_m}$
	$k_4 = \frac{E_{90}}{E_0} + \left(1 - \frac{E_{90}}{E_0}\right) \cdot \frac{a_{m-2} - a_{m-4} + \ldots \pm a_1}{a_m}$

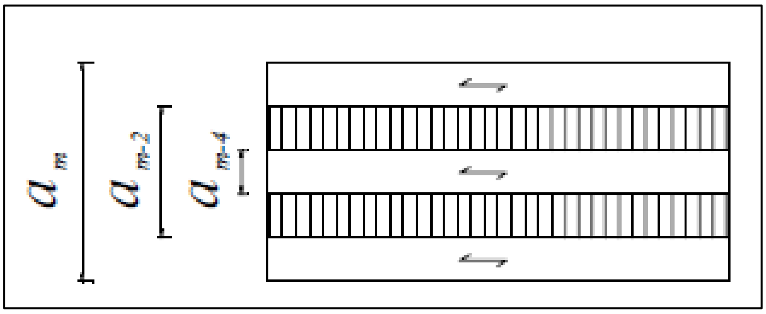


Figure 3.5: Build-up of element
Source: [28]

For the composite method, using the strength and stiffness properties of the base material for the calculation is very conservative. The timber used for the production of CLT mainly consists of boards of strength class C24. Due to the lamination effect there is a considerable improvement in properties for the element over the base material. This results in using the properties of GL28h for the calculation of an element build of boards of strength class C24. For the effective strength and stiffness properties see Table 3.4. Compared to test results these can still be considered conservative. [28]

Table 3.4: Effective values of strength and stiffness for CLT elements
Source: [28]

Loading	To the grain of outer skins	Effective strength value	Effective stiffness value
Perpendicular to the plane loading			
Bending	Parallel	$f_{m,0,ef} = f_{m,0} \cdot k_1$	$E_{m,0,ef} = E_0 \cdot k_1$
	Perpendicular	$f_{m,90,ef} = f_{m,0} \cdot k_2 \cdot a_m / a_{m-2}$	$E_{m,90,ef} = E_0 \cdot k_2$
In-plane loading			
Bending	Parallel	$f_{m,0,ef} = f_{m,0} \cdot k_3$	$E_{m,0,ef} = E_0 \cdot k_3$
	Perpendicular	$f_{m,90,ef} = f_{m,0} \cdot k_4$	$E_{m,90,ef} = E_0 \cdot k_4$
Tension	Parallel	$f_{t,0,ef} = f_{t,0} \cdot k_3$	$E_{t,0,ef} = E_0 \cdot k_3$
	Perpendicular	$f_{t,90,ef} = f_{t,0} \cdot k_4$	$E_{t,90,ef} = E_0 \cdot k_4$
Compression	Parallel	$f_{c,0,ef} = f_{c,0} \cdot k_3$	$E_{c,0,ef} = E_0 \cdot k_3$
	Perpendicular	$f_{c,90,ef} = f_{c,0} \cdot k_4$	$E_{c,90,ef} = E_0 \cdot k_4$

The effective bending stiffness can be directly determined using the modulus of elasticity:

$$(EI)_{ef} = E_0 \cdot \frac{b \cdot a_m^3}{12} \cdot k_1 \tag{3.10}$$

Which is then used to calculate the governing stresses:

$$\sigma_m = \frac{M}{(EI)_{ef}} \cdot E_0 \cdot \frac{a_m}{2} \tag{3.11}$$

3.3.2 Mechanically jointed beams theory

The Eurocode 5 (EC5) contains the rules for the design of timber structures. Cross-Laminated Timber (CLT) does not appear in EC5, however in Annex B the mechanically jointed beams theory is described.

The mechanically jointed beams theory uses an effective bending stiffness, by introducing a factor γ, that accounts for the shear deformation between layers. This theory, slightly altered, is adopted for the calculation of CLT elements. [19] [29] [30]

Effective bending stiffness:

$$(EI)eff = \sum \left(Ei \cdot \frac{1}{12} \cdot b \cdot h_i^3 + Ei \cdot \gamma i \cdot Ai \cdot ai^2 \right) \tag{3.12}$$

Where:

$$\gamma_i = \frac{1}{1+\left(\frac{\pi^2 * E0;mean * A_i * h_c}{Gr * b * L^2}\right)} \tag{3.13}$$

This calculation method can also be found in the European Technical Approval ETA-11/0189. Issued by the Deutsches Institut für Bautechnik to W.u.J. Derix GmbH & Co. [29]

The analysis, including assumptions, and calculations can be found on the next pages, followed by some background information.

The design rules for timber structures can be found in Eurocode 5. This legal document contains the requirements for design, aimed at the qualified engineer. For the background of these requirements it has to rely on the support of textbooks.

To support the Eurocode 5, the Structural Timber Education Programme (STEP) published the "Timber Engineering STEP 1". This publication contains the basis of design, material properties, structural components and joints. It is aimed to make the Eurocode 5 operational and accepted by the users.

The objective of STEP lecture B11 is to explain the computation and design of mechanically jointed beams and columns, to provide analytical solutions, and to illustrate the use of computer programs. The lecture gives inside into the background of the "mechanically jointed beams theory" (gamma method), Eurocode 5 annex B.

The computation methods described in STEP lecture B11 are based on connections made by mechanical fasteners such as nails, bolts, dowels or nail plates. In order to be able to compute reliable results it is necessary to have a continuously acting shear force along the beam. Which means that enough joints are applied and the spacing between them is equal. The displacement and force are related by the slip modulus.

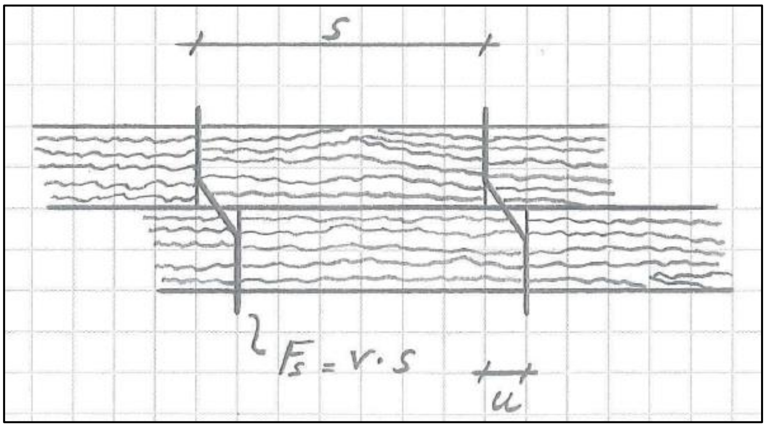


Figure 3.6: Displacement and shear force between the parts

$$v = \frac{F_s}{s}, \qquad k = \frac{K}{s}, \qquad v = k * u \tag{3.14}$$

The bending-theory is only applicable to individual components, because of the slip in the joints (due to the mechanical fasteners). The required parameters for the computation method are the stresses σ and τ, the force in the joints and the deflections. The profile and connection are regarded as continuous. Further requirements include the exclusion of the shear displacement and the simple bending-theory being valid for every part. For the system and equilibrium of an incremental element see Figure 3.7.

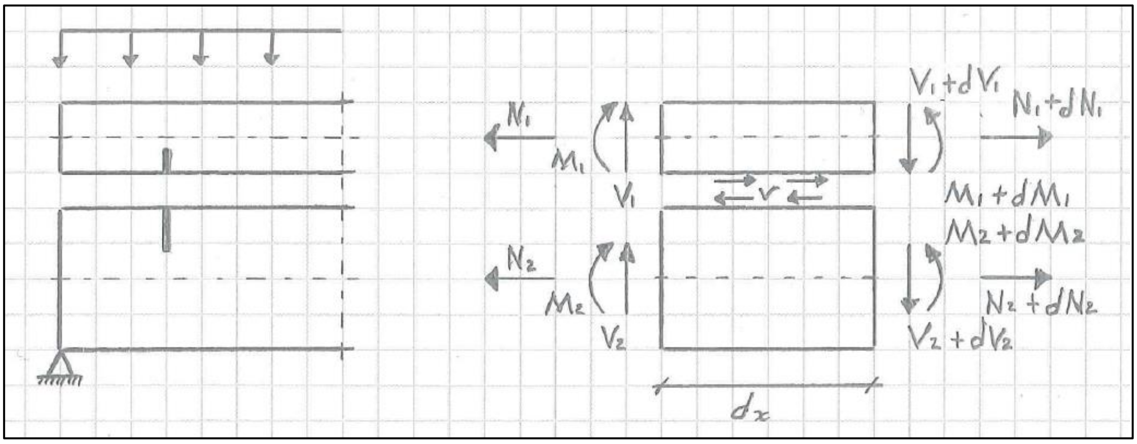


Figure 3.7: System and equilibrium of an incremental element dx

The displacement is computed from the longitudinal displacement of the individual cross-sections u_1 and u_2 and the common bending deflection w. For the deformations see Figure 3.8.

$$u = u_2 - u_1 + w'\left(\frac{h_1}{2} + \frac{h_2}{2}\right) = u_2 - u_1 + w'a \tag{3.15}$$

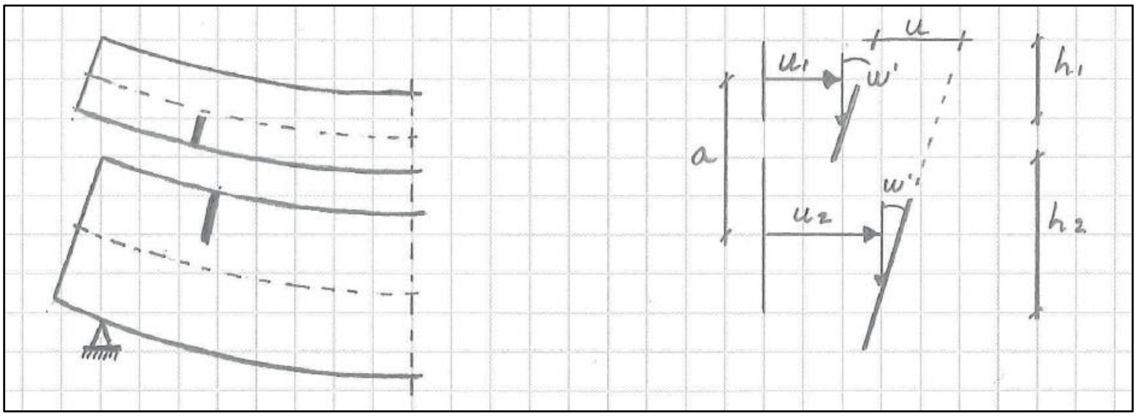


Figure 3.8: Deformations

Using equilibrium, further development of the calculation results in the equations used in EC5 Annex B.

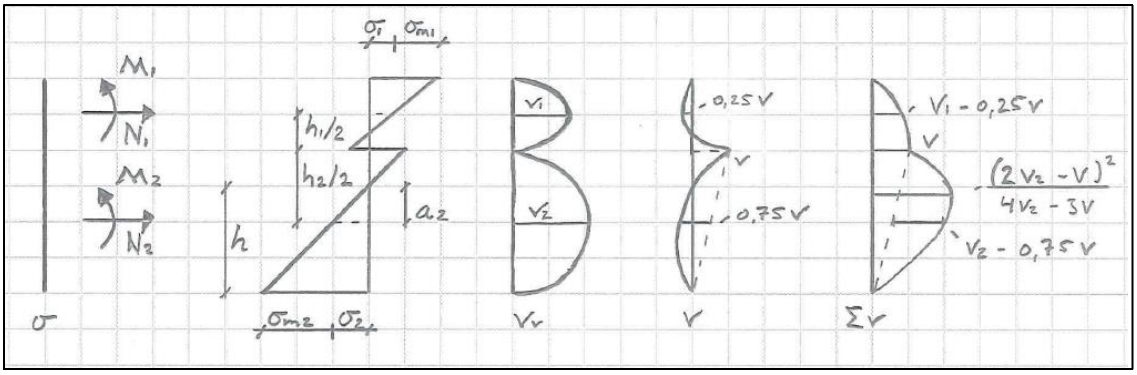


Figure 3.9: Stress distribution

For CLT the theory of mechanically jointed beams needs to be slightly altered. The cross-layers can be seen as the fasteners, which means that the rolling shear is responsible for the deformation between the longitudinal-layers.

To consider deformations due to shear the factor s_i/K_i according to EC 5 Annex B is substituted by the factor $\bar{h}_i/(G_R * b)$.

$$\gamma = \frac{u}{h} \quad ; \quad \gamma = \frac{\tau}{G} \quad with, \quad \tau = \frac{F}{s\,b} \tag{3.16}$$

$$u = \frac{F\,h}{s\,b\,G} \quad ; \quad u = \frac{F}{K} \quad \rightarrow \quad \frac{s}{K} = \frac{h}{b\,G} \tag{3.17}$$

Which leads to:

$$\gamma_i = \frac{1}{1+\frac{\pi^2\,E_i\,A_i\,s_i}{K_i\,l^2}} \quad \rightarrow \quad \gamma_i = \frac{1}{1+\frac{\pi^2\,E_i\,A_i\,\bar{h}_i}{G_R\,b\,l^2}} \tag{3.18}$$

3.3.3 Shear analogy

The shear analogy method is represented in appendix D (informative) from the German national code DIN 1052:2004-08.

The DIN 1052:2004 is the first code from Germany to define the verification process for the design of the engineered building product "Cross-Laminated Timber".

The method of the shear analogy considers the flexible connection of the layers and the shear deformation of the individual layers. For three layers with a symmetrical structure, the method is theoretically correct. In the special case of neglecting the stiffness of the individual boards perpendicular to the grain, the same is true for panels of five layers.

The code contains calculation rules for stiffness values in order to calculate forces and deformations. Stresses are calculated for the individual layers in accordance with the technical bending theory and the transverse strain is neglected. The cross-section of the structure is constructed symmetrically and the layers are aligned parallel or orthogonal to each other. For boards that are not bonded to one another on the narrow sides, the modulus of elasticity perpendicular to the grain must be set to zero.

The layers are bonded together and form a rigid composite. The stiffness is composed of a Steiner component and the bending stiffness of the individual layers. Based on the technical bending theory with consideration of shear deformation.

The composite section of CLT-panels are considered as two virtual beams, only coupled by the deflection. Beam A includes the sum of the bending stiffness of the individual layers and its shear stiffness is taken as infinite. Beam B describes the composite effect, the interaction of the individual layers of the cross-section. [31] [32] [33] [30] [34]

Effective bending stiffness:

$$(EI)eff = (EI)A + (EI)B = \sum_{i=1}^{n} Ei * \frac{1}{12} * bi * hi^3 + \sum_{i=1}^{n} Ei * Ai * zi^2 \tag{3.19}$$

The following equation is given for the shear deformation:

$$(GA)eff = \frac{a^2}{\left(\frac{h1}{2*G1*b}\right) + \sum_{i=2}^{n-1}\left(\frac{hi}{Gi*b}\right) + \left(\frac{hn}{2*Gn*b}\right)} \tag{3.20}$$

3.4 DESIGN TABLES

Design tables provided by manufacturers can be used as a starting point. The tables show possible configurations and indications based on span and loading. The design tables provided by Derix can be found in appendix: C - Design tables.

> The tables can help to plan your projects, but they do not replace structural calculations. [10]

3.5 EXAMPLES

Three configurations are calculated, using the different methods described in this chapter. This is done in order to demonstrate the design methods and to highlight the differences between configurations.

More detailed calculations can be found in appendix: D - Calculations.

3.5.1 Material properties

The elements consist of single timber boards with strength class C24. The material properties for C24 can be found in the table below.

Table 3.5: Material properties C24

Bending strength	$f_{m;k}$	24	[N/mm^2]
Tensile strength	$f_{t;0;k}$	14	[N/mm^2]
	$f_{t;90;k}$	0,4	[N/mm^2]
Compression strength	$f_{c;0;k}$	21	[N/mm^2]
	$f_{c;90;k}$	2,5	[N/mm^2]
Shear strength	$f_{v;k}$	2,7	[N/mm^2]
	$f_{r;k}$	1	[N/mm^2]
Modulus of elasticity	$E_{0;mean}$	11000	[N/mm^2]
	$E_{90;mean}$	370	[N/mm^2]
Shear modulus	G_{mean}	690	[N/mm^2]
	$G_{r;mean}$	50	[N/mm^2]
Density	ρ_k	350	[kg/m^3]
	ρ_{mean}	420	[kg/m^3]

3.5.2 Assumptions

The European technical approval ETA-11/0189 includes a table for the characteristic shear strength $f_{v;k}$. The values in this table are dependent on the thickness and number of layers of the CLT element. For these calculations the value of a single timber board with strength class C24 is used.

A strength factor is introduced for the hollow core configuration. Dependent on the number of layers at the height of the hollow core. The strength factor increases the maximum allowed bending strength. Originating from the calculations for Glulam.

Further assumptions:

- A width of 1m is used for the purpose of simplifying calculations
- Calculations are made for a simply supported single span configuration
- For the HCCLT configuration it is assumed that the effective width reaches the entire flange
- For the calculations according to the shear analogy it is assumed that an edge bond is present

3.5.3 Data

Overall dimensions:

- Width: 3.0 m
- Length: 7.2 m

Factors used for calculations can be found in the table below.

Table 3.6: Recommended factors

Modification factor	k_{mod}	0,8
Material factor	γ_M	1,3
Creep factor	k_{def}	0,9
Quasi-permanent	ψ_2	0,3
Strength factor	k_l	1,075

- k_{mod}, service class 1, according to EC 5
- γ_M, solid timber, according to EC 5
- k_{def}, according to the CLT Handbook
- ψ_2, office areas, according to EC 0

3.5.4 Contents

- Example one: 5 layer CLT-element
- Example two: 7 layer CLT-element
- Example three: 7 layer HCCLT-element

3.5.5 Dimensioning

The configuration of the examples can be found in Table 3.7. Example one and three use the same amount of material, which results in the same net cross-section. Example two and three use the same thickness for each individual layer, resulting in the same total height. These configurations are chosen to compare the result of relocating material and to investigate the influence of the hollow cores. The length of the elements is chosen as 7.2 m, since this is a common length in construction and a standard length used for hollow core concrete slabs.

Table 3.7: Configuration of the examples

	Example one	Example two	Example three
Length span [mm]	7200	7200	7200
Thickness layer 1 [mm]	40	40	40
Thickness layer 2 [mm]	20	20	20
Thickness layer 3 [mm]	40	40	40
Thickness layer 4 [mm]	20	40	40
Thickness layer 5 [mm]	40	40	40
Thickness layer 6 [mm]	-	20	20
Thickness layer 7 [mm]	-	40	40
Total height [mm]	160	240	240
Width hollow core [mm]	-	-	400
Height hollow core [mm]	-	-	120
Net cross-section [mm^2]	160000	240000	160000

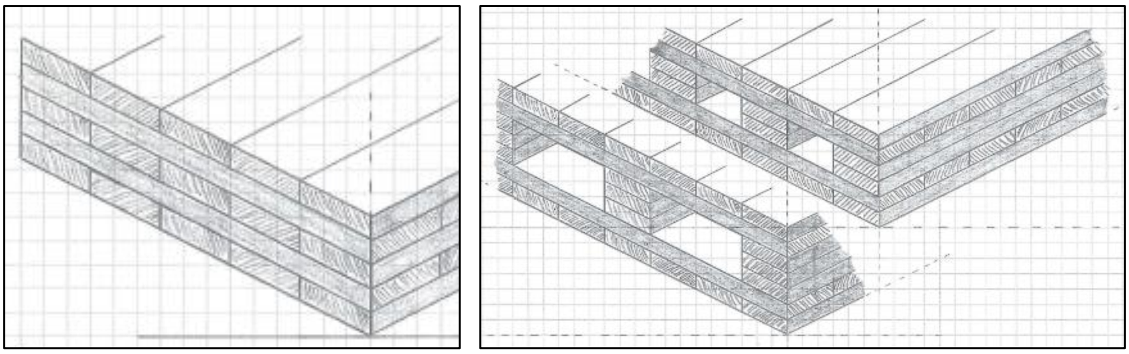


Figure 3.10: Cross-section examples

3.5.6 Loading

The loading depends on the permanent and variable loads on the element. Safety factors further determine the total design load.

The permanent loading is determined by the self-weight of the element and the loading due to floor covering.

Self-weight:

$$g_e = \rho_{mean} \cdot A \tag{3.21}$$

For:

- Example one $g_e = 0{,}672\ kN/m$
- Example two $g_e = 1{,}008\ kN/m$
- Example three $g_e = 0{,}672\ kN/m$

Permanent load due to floor covering:

- Screed $g = 1{,}0\ kN/m$

The variable loading is determined by the imposed loads, defined in Section 6 Imposed loads on buildings of Eurocode 1: Actions on structures – Part 1-1: General actions – Densities, self-weight, imposed loads for buildings. [35]

- Office areas $q = 2{,}0\ kN/m$
- Movable partitions $q = 0{,}8\ kN/m$

The design load is determined in Annex A1 Application for Buildings of Eurocode: Basis of structural design.

Design load:

$$q_d = \gamma_G \cdot (g_e + g) + \gamma_Q \cdot q \tag{3.22}$$

Where:

$$\gamma_G = 1{,}35 \tag{3.23}$$

$$\gamma_Q = 1{,}5 \tag{3.24}$$

3.5.7 Effective bending stiffness

The effective bending stiffness is calculated for examples one through three using the different design methods.

- Composite method

$$(EI)_{ef} = E_0 \cdot \frac{1}{12} \cdot b \cdot a_m^3 \cdot k_1 \tag{3.25}$$

- Mechanically jointed beams theory

$$(EI)_{ef} = \sum(Ei \cdot \frac{1}{12} \cdot b \cdot h_i^3 + Ei \cdot \gamma i \cdot Ai \cdot ai^2) \tag{3.26}$$

- Shear analogy

$$(EI)_{ef} = \sum_{i=1}^{n} Ei \cdot \frac{1}{12} \cdot b \cdot h_i^3 + \sum_{i=1}^{n} Ei \cdot Ai \cdot zi^2 \tag{3.27}$$

Table 3.8: Effective bending stiffness [Nmm^2]

	Example one	Example two	Example three
Composite method	3.358e^12	1.052e^13	9.693e^12
Mechanically jointed beams theory	3.240e^12	1.011e^13	8.632e^12
Shear analogy	3.358e^12	1.044e^13	9.445e^12

The mechanically jointed beams theory is the most conservative, with the lowest effective bending stiffness for all configurations.

For the bending stiffness of example two, the mechanically jointed beams theory makes use of two gamma factors. One for the outer layer and one for the inner layer. The "inner" layer is composed of the inner five layers and is calculated in the same manner as a five layer CLT element, like that of example one. Then with the extra gamma factor the outer layers are added to calculate the total bending stiffness of the 7 layer CLT element.

3.5.8 Normal stress

Tensile and compressive stresses are checked and should stay within the limits. Since the mechanically jointed beams theory seems the most conservative, this method is used to check the compression and tension.

- Tension check

$$\sigma_t = \frac{(M_{max} \cdot E_0 \cdot \gamma_i \cdot a_i)}{(EI)_{ef}} \leq f_{t;d} \tag{3.28}$$

- Compression check

$$\sigma_c = \frac{(M_{max} \cdot E_0 \cdot \gamma_i \cdot a_i)}{(EI)_{ef}} \leq f_{c;d} \tag{3.29}$$

3.5.9 Bending stress

Maximum bending stresses occur at the top and bottom fibres. These stresses are not allowed to exceed the critical value for bending stress.

- Composite method

$$\sigma_m = E_0 \cdot \frac{M_{max}}{(EI)_{ef}} \cdot \frac{a_m}{2} \leq f_{m;d} \tag{3.30}$$

- Mechanically jointed beams theory

$$\sigma_m = E_0 \cdot \frac{M_{max}}{(EI)_{ef}} \cdot \gamma_i \cdot a_i \leq f_{m;d} \tag{3.31}$$

- Shear analogy

$$\sigma_m = E_0 \cdot \frac{M_{max}}{(EI)_{ef}} \cdot z_i \leq f_{m;d} \tag{3.32}$$

The different design methods show small differences in the graphs for the bending stress. The γ factor of the mechanically jointed beams theory and the modulus of elasticity for the cross-layer used in the shear analogy influence the result. The graphs can be seen in the figures below.

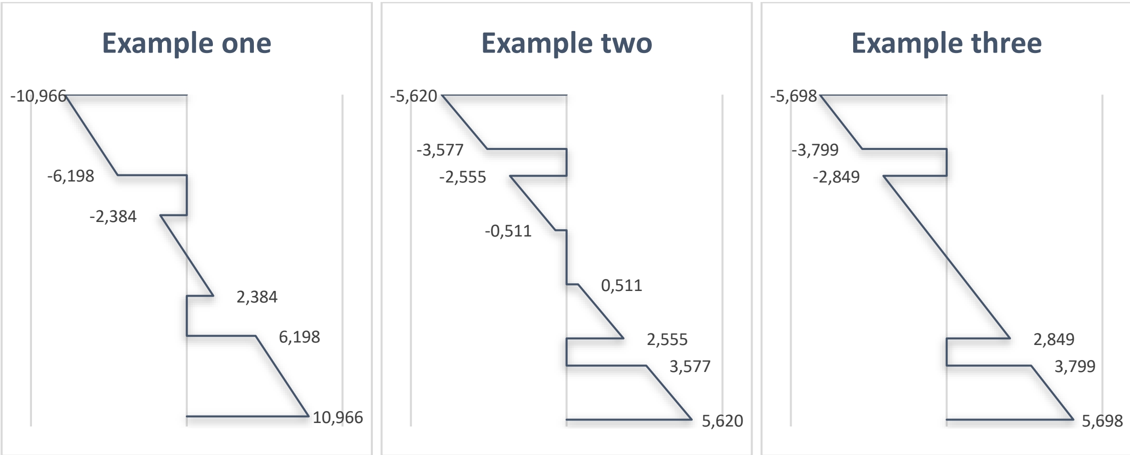


Figure 3.11: Bending stresses in N/mm², according to the Composite method

Figure 3.12: Bending stresses in N/mm², according to the Mechanically jointed beams theory

Figure 3.13: Bending stresses in N/mm², according to the Shear analogy

3.5.10 Shear stress

Maximum values for shear and rolling shear may not be exceeding critical values. Only the mechanically jointed beams theory and the shear analogy perform calculations on shear.

- Composite method

N.A.

- Mechanically jointed beams theory

$$\tau_v = V_{max} \cdot \frac{(E_0 \cdot A_1 \cdot \gamma_1 \cdot a_1)}{(EI)_{ef} \cdot b} \leq f_{v;d} \tag{3.33}$$

- Shear analogy

$$\tau_v = V_{max} \cdot \frac{E \cdot S}{(EI)_{ef}} \leq f_{v;d} \tag{3.34}$$

The boards oriented in longitudinal direction are stressed in longitudinal shear, the values can be found in the figures below. For the boards oriented in transverse direction these values are that of rolling shear, τ_r. The methods show a very similar distribution of stress.

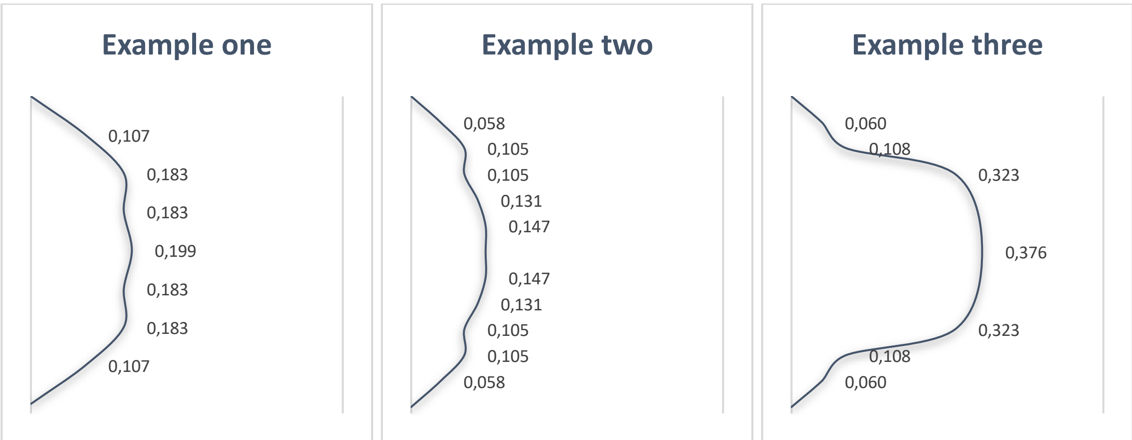


Figure 3.14: Shear stresses in N/mm², according to the Mechanically jointed beams theory

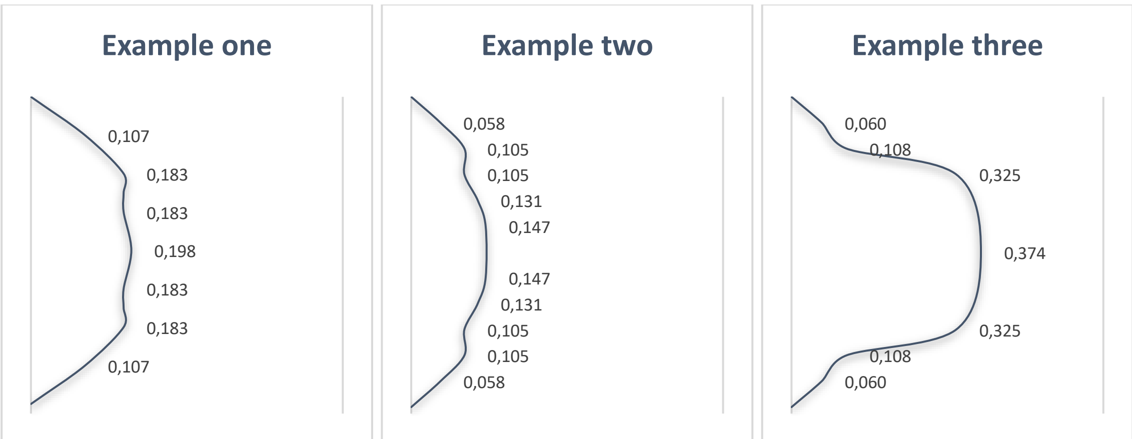


Figure 3.15: Shear stresses in N/mm², according to the Shear analogy

Extra checks are performed on the hollow core configuration. The longitudinal shear stress in the cross-layer, that occurs due to the cooperating flanges, is checked. As well as the bending and shear stresses for the cross-span over the hollow cores.

The force in the longitudinal oriented boards over the hollow cores needs to be transferred to the web. Since the boards are not edge bonded the cross-layer is responsible for the shear of area $A_{longitudinal}$, only the height of the cross-layer, h_{cross}, is stressed.

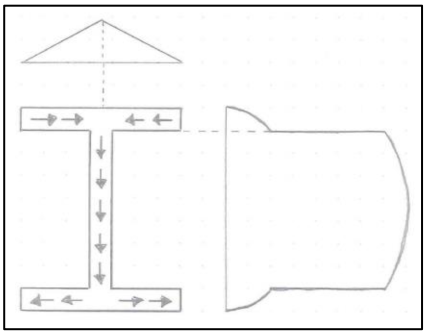


Figure 3.16: Shear distribution I-profile

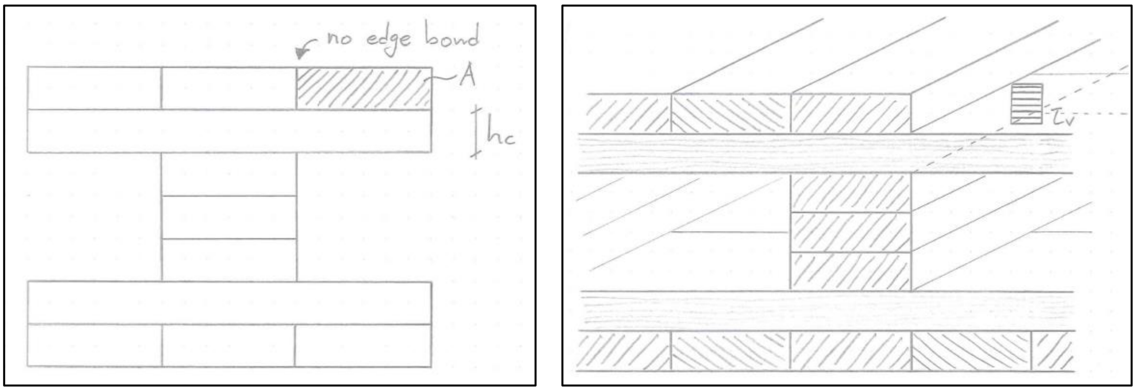


Figure 3.17: Longitudinal shear stress in the cross-layer, due to cooperating flanges

The check is performed where flange and web meet. The longitudinal shear in the cross-layer is calculated with the equation below, this is done according to the Mechanically jointed beams theory.

$$\tau_v = \frac{V_{max} \cdot (E_0 \cdot A_{longitudinal\,span} \cdot \gamma_1 \cdot a_1)}{(EI)_{ef} \cdot h_{cross\,layer}} \leq f_{v;d} \tag{3.35}$$

Additional bending and shear checks are performed for the cross layer, spanning the hollow core. Simplified calculations are used.

- Bending

$$\frac{M}{W} \leq f_{m;d} \tag{3.36}$$

- Shear

$$\frac{1{,}5 \cdot V}{A} \leq f_{v;d} \tag{3.37}$$

3.5.11 Deformation

The total deformation can be divided into the deformation due to permanent and variable loading. Additional factors for creep and time further determine the final deflection.

The deformation is calculated with the effective bending stiffness found with the Mechanically jointed beams theory, since this is the most conservative one.

$$w = \frac{5}{384} \cdot \frac{q \cdot l^4}{EI} \tag{3.38}$$

The final deformation is calculated with the simplified equations from Section 2: Basis of design of the Eurocode 5 – Design of timber structures – Part 1-1: General – Common rules and rules for buildings. This is done under the assumption that the factor for creep, according to the CLT Handbook, will suffice.

$$u_{fin} = u_{fin,G} + u_{fin,Q1} + u_{fin,Qi} \tag{3.39}$$

where:

$$u_{fin,G} = u_{inst,G}\,(1 + k_{def}) \tag{3.40}$$

$$u_{fin,Q1} = u_{inst,Q1}\,(1 + \Psi_{2,1}\,k_{def}) \tag{3.41}$$

Table 3.9: Instantaneous and final deflection

	Example one	Example two	Example three
Instantaneous deflection [mm]	48.3	16.6	18.1
Final deflection [mm]	72.7	25.5	27.3

Limiting values for the deformation are taken according to Table 7.2 – Examples of limiting values for deflections of beams from Eurocode 5 – Design of timber structures – Part 1-1: General – Common rules and rules for buildings.

$$w_{inst} \leq l/300 \text{ to } l/500 \tag{3.42}$$

$$w_{fin} \leq l/250 \text{ to } l/350 \tag{3.43}$$

The Shear analogy includes the deformation due to shear, however due to the fact that an edge bond is assumed for the Shear analogy the additional deformation is minimal. Roughly 1 to 2 mm.

$$w = \frac{5}{384} \cdot \frac{q \cdot l^4}{EI} + \frac{q \cdot l^2}{8 \cdot GA} \tag{3.44}$$

3.5.12 Vibration

The Eurocode 5 includes design criteria that apply to a fundamental frequency higher than 8 Hz. Floors with a lower fundamental frequency require special investigation.

The requirements in Eurocode 5 are for a rectangular floor with overall dimensions $l \cdot b$, simply supported along all four edges and with timber beams spanning in l direction. An approximation of the fundamental frequency can be calculated with the equation below.

- Fundamental frequency

$$f_1 = \frac{\pi}{2 \cdot l^2} \cdot \sqrt{\frac{EI}{m}} \tag{3.45}$$

The mass m in kg/m^2 is taken as the permanent load only, partitions and variable loading are excluded.

Table 3.10: Fundamental frequencies

	Example one	Example two	Example three
Fundamental frequency [Hz]	4.2	6.8	6.9

For residential floors with a fundamental frequency greater than 8 Hz the following requirements should be satisfied:

$$\frac{w}{F} \leq a\ [mm/kN] \tag{3.46}$$

$$v \leq b^{(f_1 \zeta - 1)}\ [m/(Ns^2)] \tag{3.47}$$

w is the maximum instantaneous vertical deflection due to a vertical concentrated static force F applied at any point on the floor.

v is the maximum initial value of the vertical vibration velocity produced by an ideal unit impulse of 1 Ns, applied at the point on the floor giving the maximum response. Components above 40 Hz may be disregarded.

ζ is the modal damping ratio. Unless other values are proven to be more appropriate, a value of 1 % should be assumed.

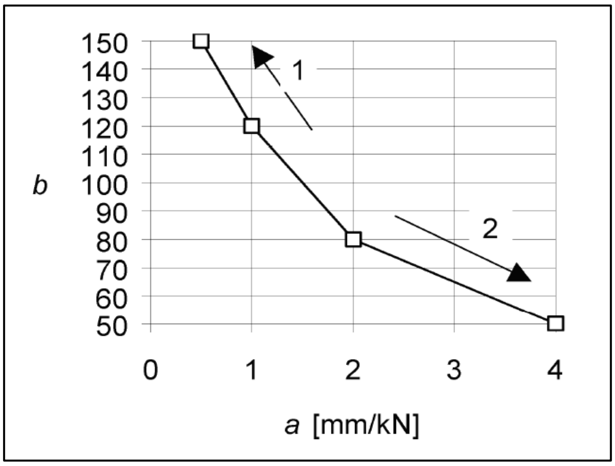


Figure 3.18: Recommended range of and relationship between a and b
Source: [19]

Figure 3.18 shows the recommended range of the relationship between a and b, 1 better performance, 2 poorer performance. A quick calculation for the HCCLT element showed a value of approximately 1 for a and roughly 1200 for b, placing the outcome in the better part of the recommended range. For this calculation assumptions were made regarding the width of the floor, bending stiffness in cross-direction, the deformation, etc.

3.6 CONCLUSION

This chapter covered the structural design of cross-laminated timber. CLT is a relative flexible building material with a low dead weight. The design is often determined by serviceability criteria, such as deformation and vibration. Strength criteria, like bending and shear are mostly not governing. However, the rolling shear properties of the cross-layers can control the design, it influences the effective bending stiffness and stress distribution. It also causes a larger deformation due to shear than for other wood based products.

The calculations performed in this chapter show the deformation governing the design for all three configurations. With the different methods all showing a very similar outcome, while the composite method does not account for shear deformation and both the mechanically jointed beams theory and the shear analogy do. This is mainly due to the large span over depth ratio, reducing the influence of the shear deformation. The mechanically jointed beams theory from the EC 5 is the most conservative, showing the lowest effective bending stiffness.

Results for the normal stress, due to bending, and the shear stress are also very close. Again with the EC 5 as the most conservative, showing the highest values, due to the lower effective bending stiffness. For the normal stress the differences between configuration two and three show little influence by the reduction in nett cross-section. The HCCLT configuration does show larger values for shear. Because of the hollow cores there is less material in the middle of the element, resulting in shear stresses between 2 to 3 times higher compared to the other configurations. Still the values are well within the limits.

An extra shear check is performed on the HCCLT configuration, in this case not governing the design. However, the values are such that it may influence the thickness of the cross-layers for other specific cases and should be looked into.

The vibrations are assumed not governing the design, showing first fundamental frequencies of approximately 7 Hz for examples two and three. The already small contribution of the dead weight to the loading is not altered much by the reduced net cross-sectional area of the HCCLT configuration and as a result does not significantly affect the fundamental frequency. In combination with the amplitude of acceleration the behaviour does however show a negative effect on the comfort of the user.

The possible size of the hollow cores depends on the length of the span, loading situation and element configuration. In the case of these examples a reduction of more than 30% for the net cross-sectional area is realised with the hollow cores. The effective bending stiffness is almost the same for examples two and three and when comparing example one to example three, which uses the same amount of material, the effective bending stiffness is greatly increased, showing a much more efficient use of material.

4 MODELLING & STRUCTURAL ANALYSES

The behaviour of Hollow Core Cross-Laminated Timber, both during manufacturing and in use, is unknown. The calculations in the previous chapter show little difference between examples, with respect to the capacity. This would indicate that a more efficient use of material is possible.

Multiple different models are made and analysed in this chapter, in order to investigate the behaviour and determine the influence of the manufacturing process on the possible composition of HCCLT.

To produce the 3D mechanical models, Autodesk Inventor 3D CAD software is used. This software then allows you to connect with and analyse the model in ANSYS.

4.1 MODELLING

Derix has produced test samples of 6- and 7-layer HCCLT elements. The two different configurations can be seen in Figure 4.1. The test samples were produced by pausing the manufacturing process in order to relocate the boards of a single layer to form the web. For the purpose of these test samples this was done by hand, however this procedure can easily be automated for production purposes.

Figure 4.1: Hollow Core CLT-element
Source: W.u.J. Derix GmbH & Co

4.1.1 First attempt

As a first attempt a model was made using the layout of one of the test samples made by Derix. A "full" HCCLT element, measuring 3,0 m in width and spanning 7,2 m in length was modelled.

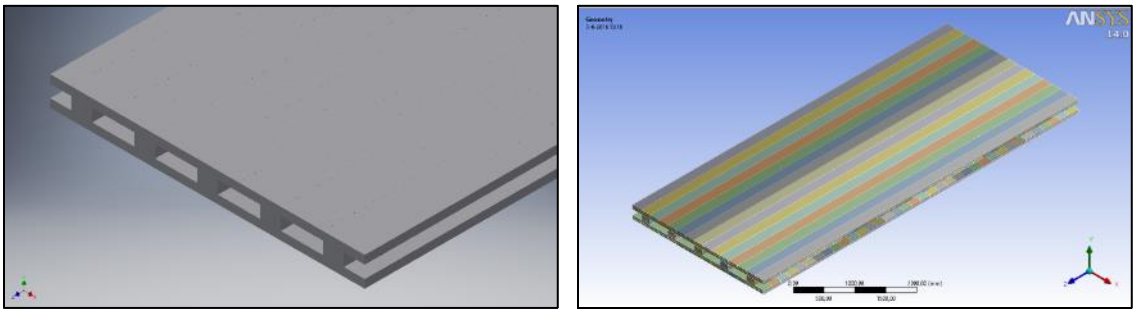


Figure 4.2: "Full" HCCLT-element modelled in Autodesk Inventor

The default for all connections is bonded, this would suggest that all edges are glued and a proper bond is realized all around. This is however not the case, many companies including Derix do not glue the narrow faces of the individual boards. This means that there is no edge bond present. To account for this the connections between boards in a single layer have to be modelled differently, for this first attempt the connections are simply suppressed. This means that the elements can move freely, there is no connection realized between the sides of the boards.

The deformation and equivalent stress distribution can be seen in Figure 4.3 and Figure 4.4. The result of suppressing the connections on the narrow faces can be observed in the stress distribution of the cross layers. It clearly shows the introduction of stress for each individual board, all boards are individually stressed.

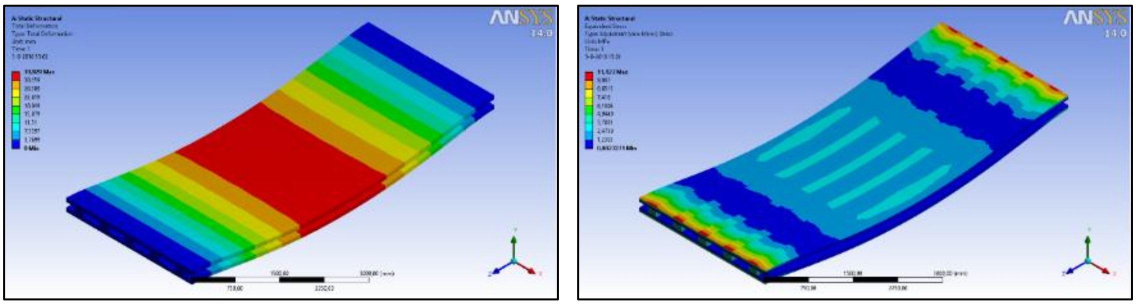


Figure 4.3: Deformation and equivalent stress

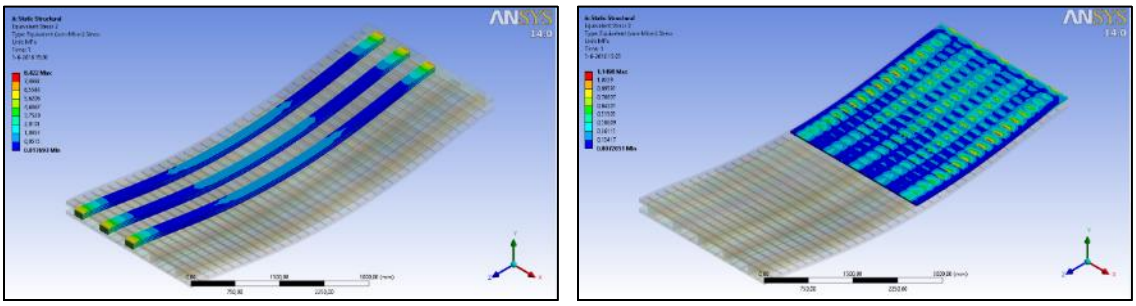


Figure 4.4: Equivalent stress – web and cross layer

This first attempt showed the possibility of modelling the HCCLT element. However, the size of the model limits model refinement, with respect to mesh, connection options, etc. A closer look into the types of connections and contact options is necessary in order to use the appropriate options to further refine the model.

4.1.2 Review on modelling options

4.1.2.1 Type of connection

The imported geometry automatically determines all connecting surfaces as being bonded, however not all connecting surfaces are bonded. The narrow faces are not glued and during manufacturing the glue is still wet for the wide faces, so does not yet provide any bond. The different options for the connection between surfaces are bonded, no separation, rough, frictionless and frictional.

The bonded connections are not allowed to separate and slide, the surfaces are bonded like glue. Tolerance in the design assures that surfaces will be together, with or without the presence of a gap and or in the case of penetration. No separation allows for the surfaces to slide a bit, relative motion is allowed. A rough connection cannot slide but is allowed to separate. Frictionless connections can separate and slide freely, with the friction coefficient set to zero. Compared to the frictionless connection option, the frictional connection option uses a user defined coefficient of friction. The surfaces will only slide after a certain threshold resistance value for the shear stress, which depends on the coefficient that is chosen.

4.1.2.2 Contact options

When the surfaces of two separate elements touch, such that they become mutually tangent, they are in contact with each other. Contact between surfaces is usually described by the following characteristics: The surfaces do not interpenetrate and they can transmit compressive normal forces and tangential friction forces. However, they often do not transmit tensile normal forces, which means that the surfaces can separate and move away from each other.

To enforce impenetrability, prevent the surfaces from passing through each other in the analysis, a relationship must be established between surfaces. Several algorithms can be chosen to enforce contact compatibility.

The contact algorithms Pure Penalty and Augmented Lagrange are both penalty-based. The formulation of these algorithms is based on the contact force F_{normal}, the contact stiffness k_{normal} and the penetration $x_{penetration}$. The higher the contact stiffness the lower the penetration. For penalty-based methods it is not possible to achieve zero penetration, but the solutions will be accurate as long as the penetration can be considered negligible.

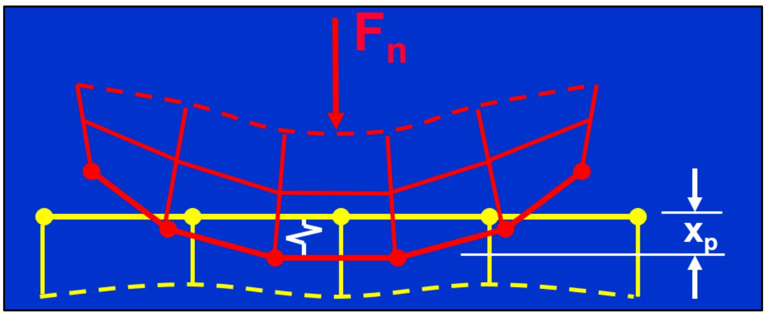


Figure 4.5: Visualization of penalty-based algorithms
Source: [36]

Pure Penalty:

$$F_{normal} = k_{normal} \cdot x_{penetration} \tag{4.1}$$

Augmented Lagrange:

$$F_{normal} = k_{normal} \cdot x_{penetration} + \lambda \tag{4.2}$$

Both accuracy and convergence behaviour are affected by the normal contact stiffness. A larger stiffness means a higher accuracy, but can lead to difficulties in the convergence, which may cause the model to oscillate. Because of the extra term present in the Augmented Lagrange method, it is less sensitive to the contact stiffness.

An alternative to the penalty-based methods is the Lagrange algorithm. The normal Lagrange algorithm contains an added degree of freedom, introduced by solving the contact force as an DOF instead of a function of the contact stiffness and the penetration, to satisfy contact compatibility. The advantage of this method is the zero to nearly-zero penetration and does not require contact stiffness. The computation can be more demanding. Another disadvantage can occur in the form of chattering which often occurs if no penetration is allowed. The contact between surfaces becomes a step function, this can lead to oscillating between the contact status being open or closed and makes the convergence difficult.

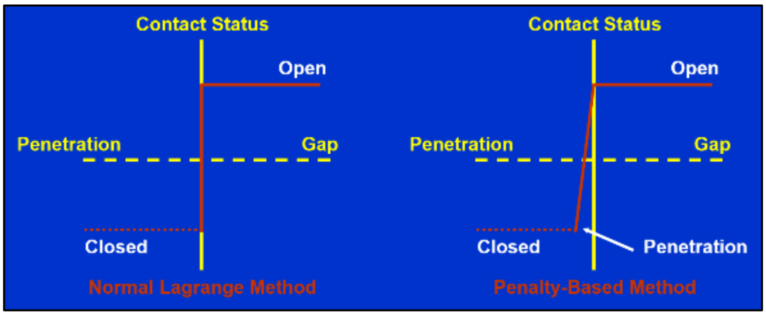


Figure 4.6: Normal Lagrange as an alternative to Penalty-Based
Source: [36]

Finally the Multi-Point Constraint (MPC) formulation is described. This algorithm can be used in the case of bonded contacts only. It adds constraint equations to the displacement between surfaces. Advantages are its good convergence, no required contact stiffness and no penetration. It's an efficient way of relating surfaces of bonded connections, not penalty-based or Lagrange multiplier-based.

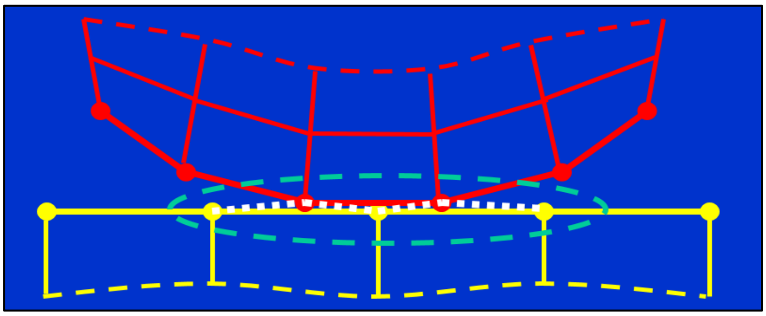


Figure 4.7: Multi-Point Constraint, for bonded contacts only
Source: [36]

4.1.2.3 *Edge bonded or non-edge bonded*

Individual boards are connected to each other over the wide faces by gluing. The narrow faces may either be in contact, with or without being glued, or may be arranged with a small spacing between each other.

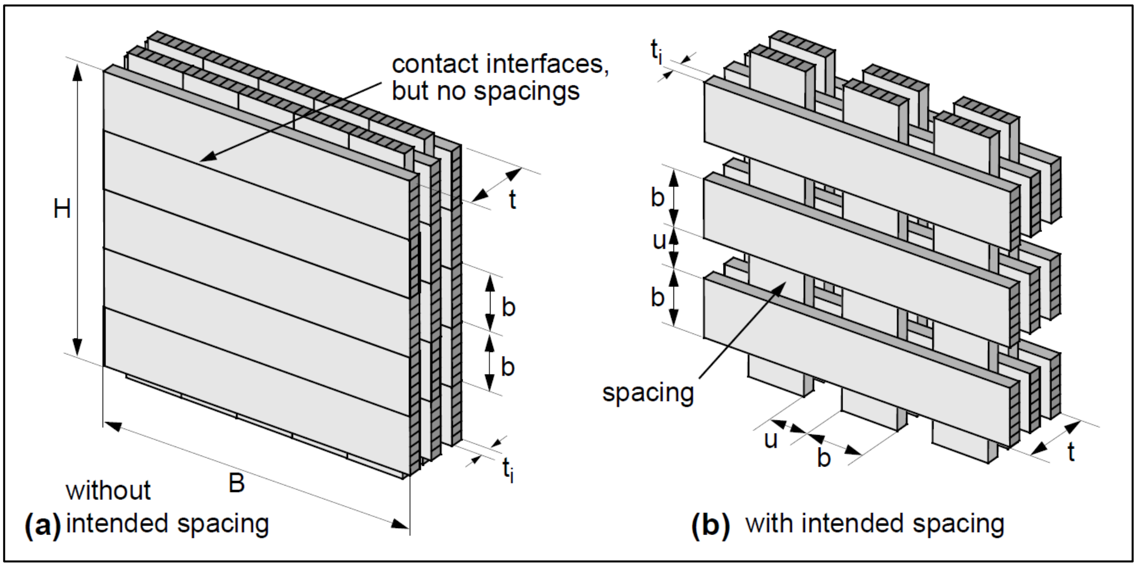


Figure 4.8: CLT without being glued at the narrow faces, with and without intended spacing
Source: [37]

Considering the standard configuration of CLT elements, which means no intended spacing between the narrow faces and without gluing the narrow faces, no edge bond. Modelling the connection between boards at the narrow faces can be done by using a contact options that allows for some movement between boards or the contact between boards can be neglected by suppressing the contact option. Unintended spacing due to shrinkage or swelling make the coefficient of friction and contact conditions uncertain, even for configurations that are glued at the narrow faces. [37] [38]

The connection between layers is realised by gluing the wide faces of the boards together. This connection is modelled as bonded, which provides a rigid connection. The connection is not allowed to separate or slide.

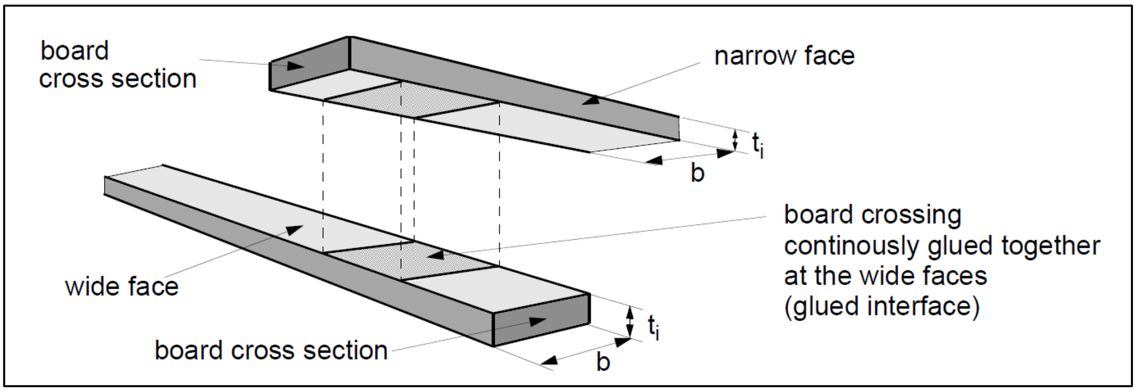


Figure 4.9: Glued joint of a single board crossing
Source: [37]

Adjacent laminations may be edge bonded or non-edge bonded. For non-edge bonded laminations, the width of gaps between adjacent boards within a timber layer shall be less than or equal to 6 mm [27].

Figure 4.10: HCCLT scale model, non-edge bonded with exaggerated gaps

4.2 DURING MANUFACTURING

An option to produce Hollow Core Cross-Laminated Timber is separately pressing the flanges and the web. Producing CLT panels and Glulam beams and then combining them. This is however time consuming and to more efficiently produce HCCLT elements, the possibility of pressing the whole element at ones will be investigated.

At Derix, CLT elements are pressed in a hydraulic press with a pressure of 0.6 MPa. The expected distribution of stress for the configuration seen in Figure 4.11 results in an average stress of 1.8 MPa for the web, with peak stresses in the corners where the web and flange meet. Stresses in the flanges might be minimal, due to the possibility of "free" motion in vertical direction.

These stresses might result in glue squeeze out in area one and insufficient pressure to properly bond layers in and around area two, see Figure 4.11.

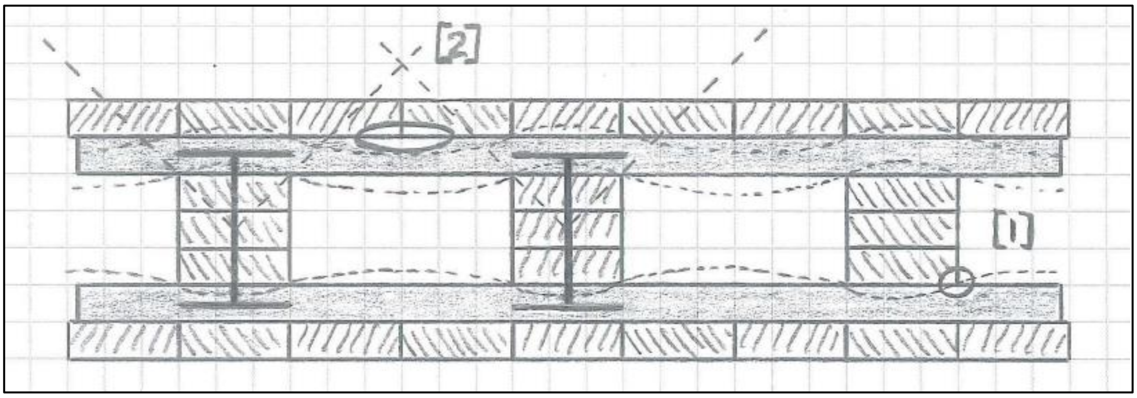
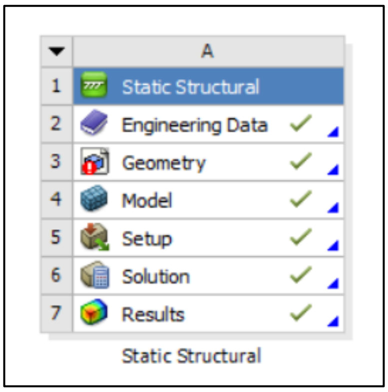


Figure 4.11: Problem areas for manufacturing, Hollow Core Cross-Laminated Timber

4.2.1 Engineering data



The material library of ANSYS does not include timber properties. The engineering data used for the analysis can be found in Table 3.1. Since timber is anisotropic, two new materials are added to the library, Timber C24 and Timber C24 Transverse (the latter is used for the cross layers).

4.2.2 Geometry

The models made in Autodesk Inventor are imported into the geometry of ANSYS for analysis. For the purpose of analysing the behaviour during pressing the models are limited to a length of 1,0 m. Two models are made and analysed, a model with an II-profile and by making use of symmetry an R-profile. The II-profile is used to determine the behaviour and to check the outcome of the R-profile. By limiting the length of the models and reducing its size, for the R-profile, more freedom is provided for choosing mesh size and other options that limit computer calculations, such as connection options. The model with the R-profile is therefore used to perform most of the analysis.

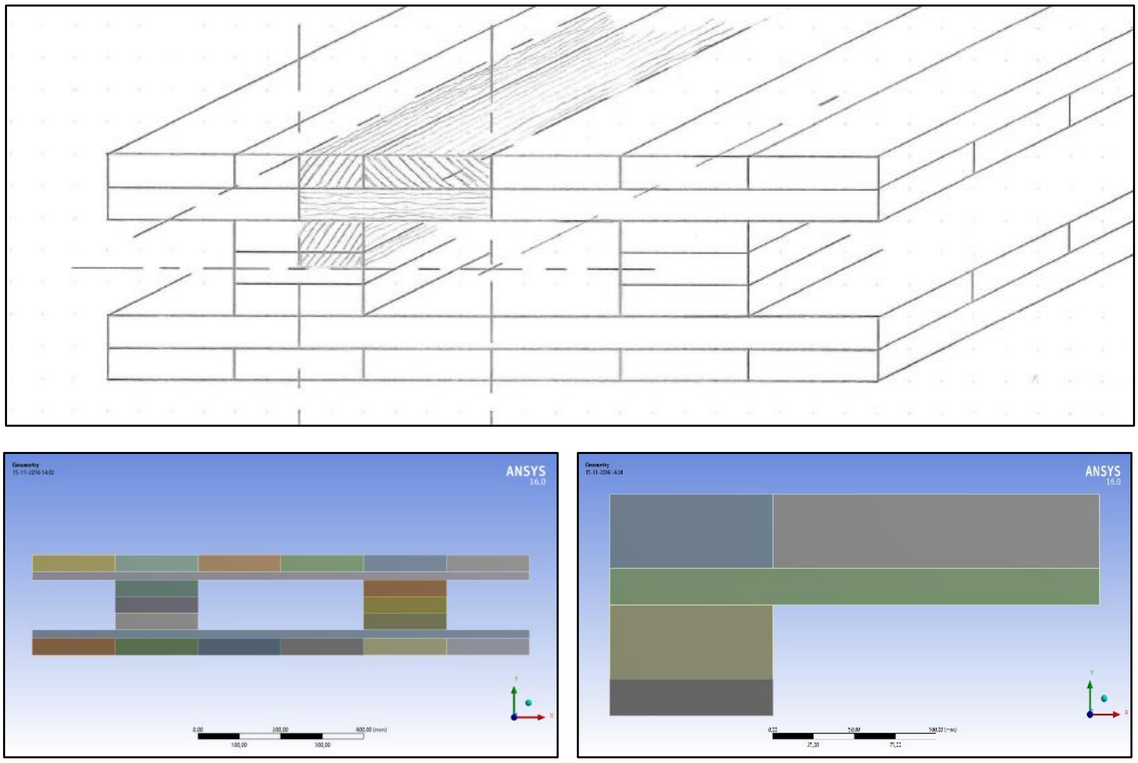


Figure 4.12: Models with an II-profile and, by symmetry, an R-profile

4.2.3 Model

4.2.3.1 Connections

During manufacturing the elements are glued and pressed in order to produce a bond between layers. This means that when the elements are in the process of being pressed there is no bond present. In order to be able to model the behaviour different connection options are chosen for the different contact regions.

The contact regions are separated into two categories. Connections between the wide faces and narrow faces of the individual boards.

Since there is no edge bond present, the side by side contact between boards is suppressed. Compared to using the frictionless contact option the computing time and power needed are much lower and still provide accurate results provided the following holts: It should be checked that the elements representing the individual boards do not significantly crossover / overlap, since this is not possible and may influence the outcome of the analysis.

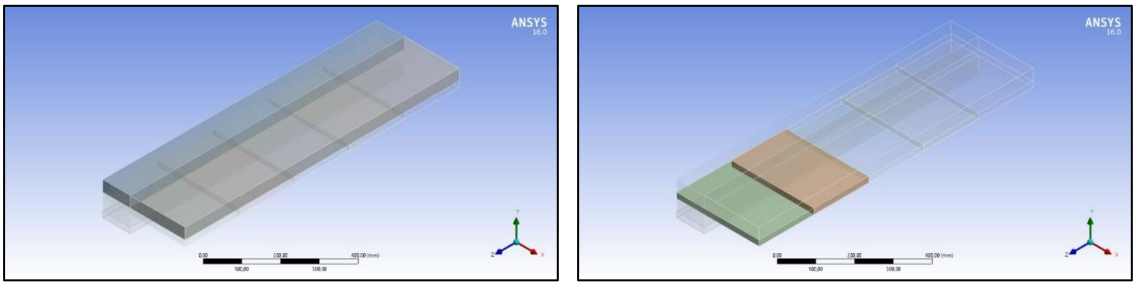


Figure 4.13: Suppressed contact regions

All contacts on the wide faces are modelled as bonded. In reality this is not the case since the element is in the process of being pressed and a bond is hopefully realised during this procedure. However by modelling the contact as bonded the results of the normal stress in y-direction can be checked to see if the pressure is transferred from one layer to another. And if tensile stresses occur between layers it shows that the layers will separate.

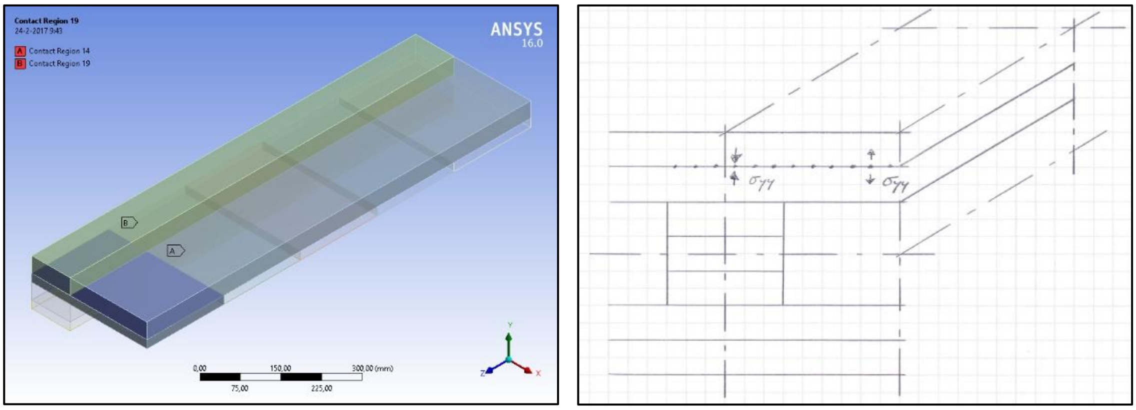


Figure 4.14: Bonded contact region and proposed bond check

4.2.3.2 Mesh

Many different mesh shapes and sizes can be chosen to perform the analysis. And these can be further refined at specific locations, edges etc. The following mesh is used to provide accurate results and maintaining an even distribution over the whole element, which in turn also contributes to the accuracy of the results. [39]

For both the II-profile and R-profile a square mesh is taken. This shape best suits the situation, because of its even distribution and symmetry over the model. Limits for the number of elements determined sizing. Resulting in a 10x10x10 mm^3 mesh for the II-profile and a 5x5x5 mm^3 mesh for the R-profile.

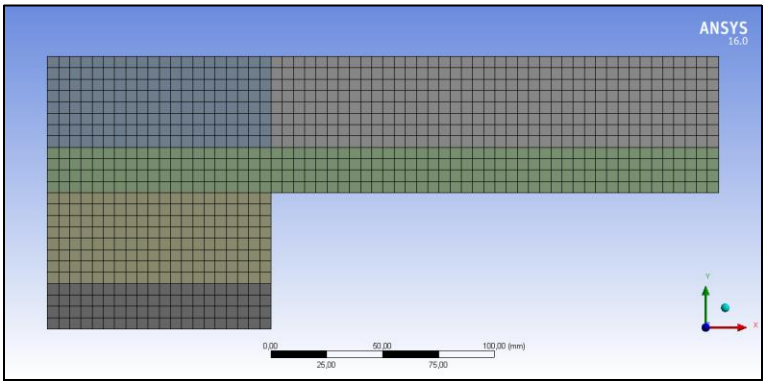


Figure 4.15: Mesh distribution

4.2.4 Setup

A hydraulic press is used for the manufacturing. Applying a pressure on the top surface of the model does not have the desired outcome. Since the flanges of the element are less resistant to the applied pressure the deformation on the top surface is uneven and not reliable or true to reality. To model the pressing process of an element, displacement restrictions are used.

Three different setups are uses for the models. Pressing the element without supporting the flanges, and with partial and full support structures in the hollow cores. This is done in relation to the expected problems described in the beginning of this chapter.

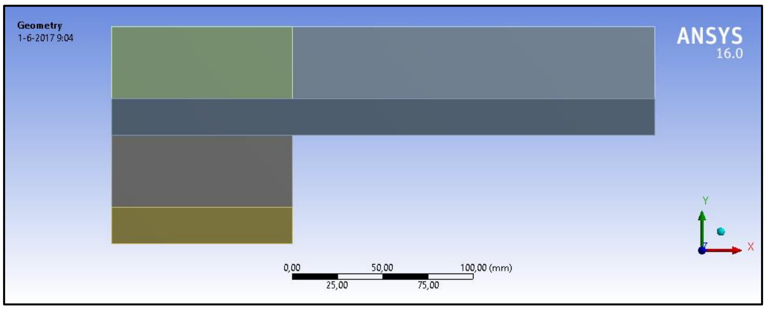


Figure 4.16: No support structure

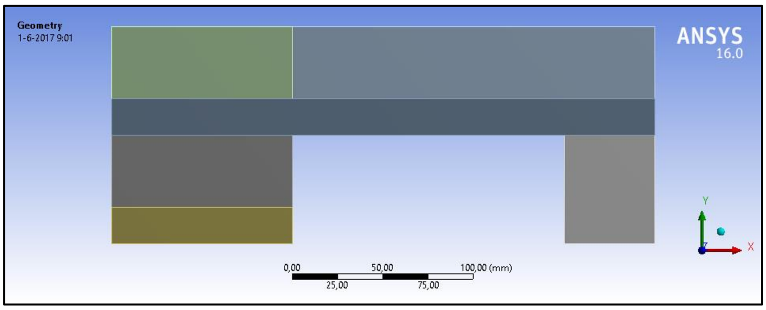


Figure 4.17: Partial support structure

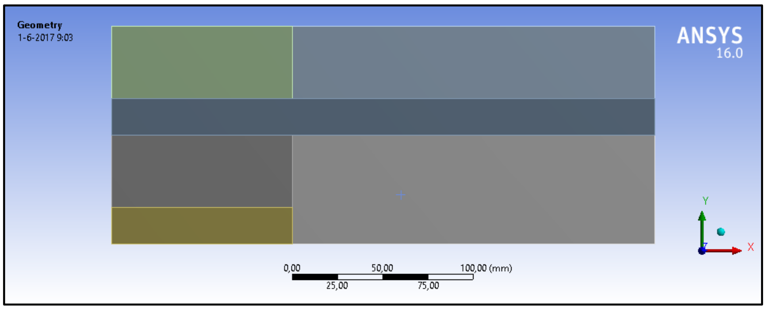


Figure 4.18: Full support structure

4.2.4.1 Displacements & frictionless supports

To model the behaviour of the pressing process the bottom surface of the element is restricted in moving in vertical direction, while the top surface is forced down by a certain distance, corresponding to the pressure produced by the press. This distance can be determined with the modulus of elasticity and is checked in the analysis.

The zero displacement restriction on the bottom surface is the same for both the II-profile and the R-profile. Because of the symmetry around the x-axis the height of the model is reduced to half and so is the displacement restriction placed on the top surface of the model.

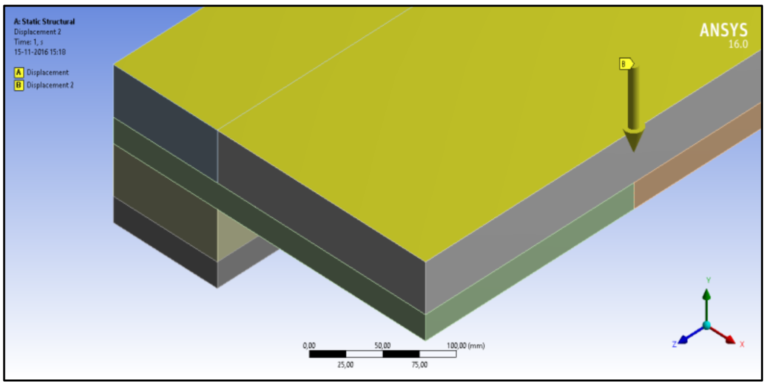


Figure 4.19: Displacement restrictions

The R-profile requires boundary conditions for symmetry. These conditions can be enforced with frictionless supports on the left and right surfaces. On the right side only the cross layers are supported by frictionless supports, since there is no bond present at the narrow faces of individual boards. Figure 4.20 shows the frictionless supports, in blue.

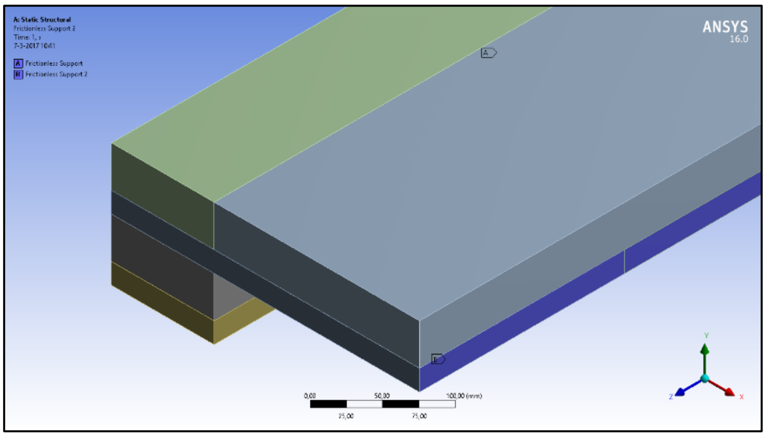


Figure 4.20: Frictionless supports

4.2.5 Solution and results

Both models are analysed with and without support structures located in the hollow cores. A support in the middle of the span is tested as well as a support spanning the full length of the hollow core.

4.2.5.1 Force reaction

Displacement restrictions are used to simulate the behaviour of the press. A force reaction on the bottom surface is used to verify the pressure realized by the displacement restrictions. The force reaction is a product of the pressure and surface area and since we know the surface area, we can use the force reaction to determine the pressure.

The configuration of the element results in an uneven distribution of stress. Introducing a pressure of 0,6 MPa over the whole top surface resulted in unnecessarily high stresses in the web. To determine the displacement restrictions the pressure is checked there where the net cross sectional area is minimal, in order to not introduce unnecessarily high stresses.

The displacements introduced on the top surface are 0,39 mm for the model with the II-profile and 0,195 mm for the R-profile. The modulus of elasticity is used to determine the needed strain in y-direction. The force reaction resulted in nearly the same displacements.

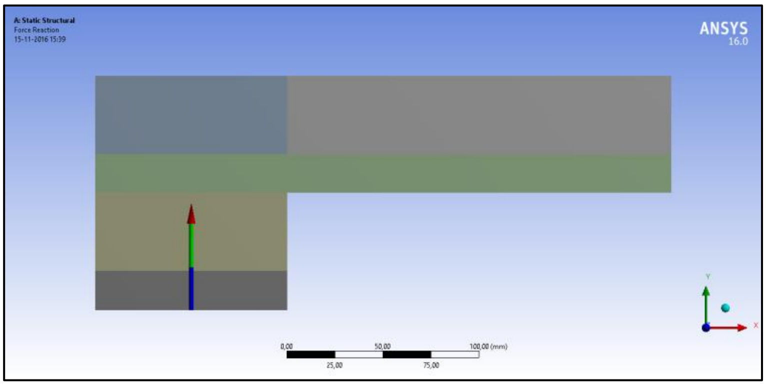


Figure 4.21: Force reaction

4.2.5.2 Deformation

The displacement restrictions for the configuration without any additional supports result in the deformation that can be seen in Figure 4.22. Showing maximum deformation at the top and over almost the whole width of the flange.

The flange shows little resistance to the deformation. It can be observed that the deformation in the flange is similar over most of its width, it only differs very close to the web. Here the web and part of the transverse layer provide support and resistance to the deformation.

In Figure 4.22, showing the deformation of the R-profile, a discontinuity in deformation can be observed, located at the narrow faces of the boards in the top layer. This is due to the suppressed contact region, accounting for the lack of edge bond.

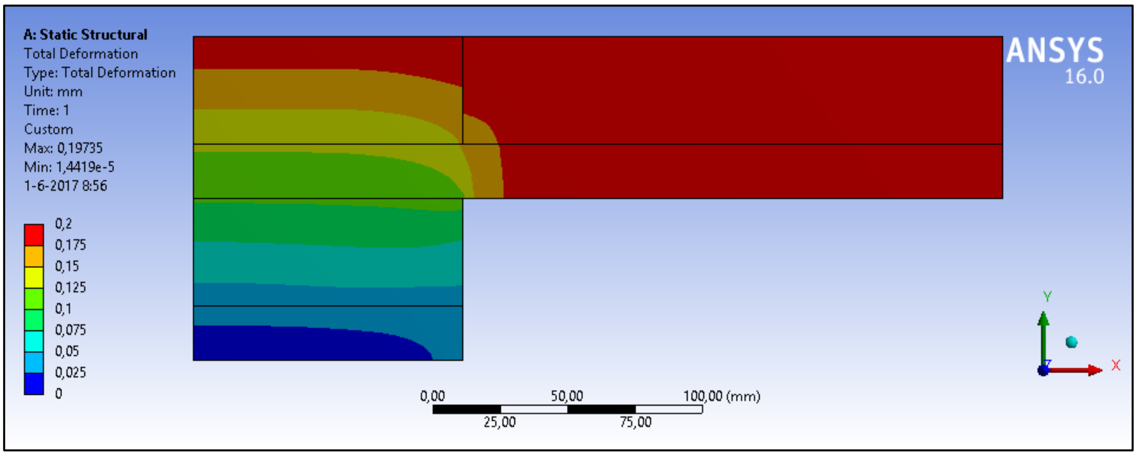


Figure 4.22: Deformation

The two configurations where the flange is supported during pressing show improvement. The maximum deformation occurs over a smaller area. The partial support mainly influences the deformation close to the supported area. The location underneath of where two boards in longitudinal direction meet shows some improvement in distribution. Supporting the whole span shows an even deformation, with minor disruptions at the sides of individual boards.

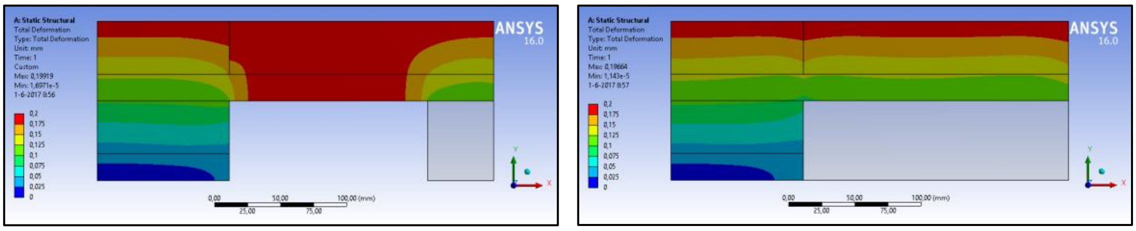


Figure 4.23: Deformation, partially and fully supported

4.2.5.3 Normal stress

For the distribution of normal stress in y-direction the same can be observed as for the deformation. Since the unsupported flange is moving relatively freely in vertical direction, which is the direction of pressure, it is almost not stressed. This means that the contribution of the flange is small and that almost all pressure is taken by the web.

Since most of the pressure is transferred directly through the web a lower pressing pressure is used on the top surface. Providing a solution for the peak stresses located at the corners, which values where too high and protruded too far into the material to assume a sufficient redistribution of stresses. However, still not offering a solution for the flanges. The normal stress distribution in y-direction, shown in Figure 4.24, shows no compression in the flange. The middle of the flange even shows some tension, indicating the layers in the flange would separate.

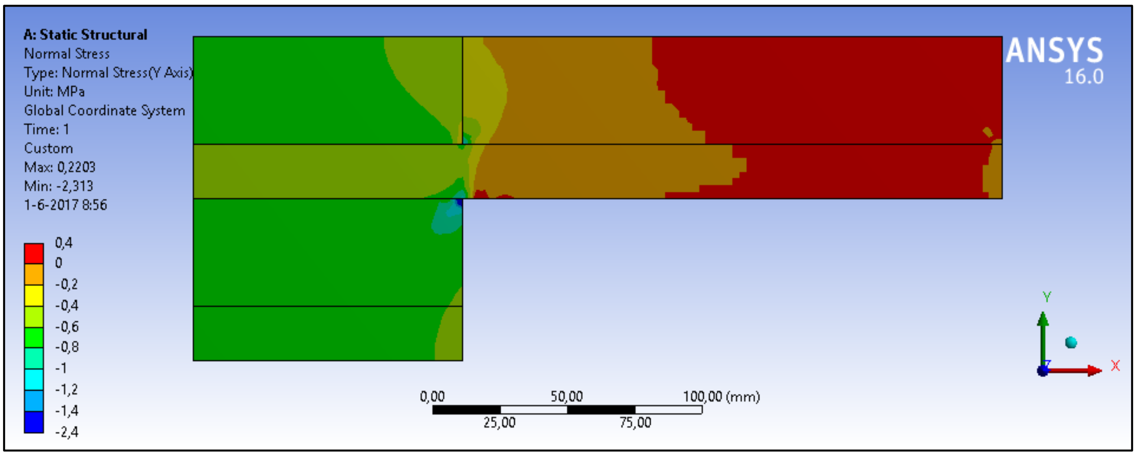


Figure 4.24: Normal stress in y-direction

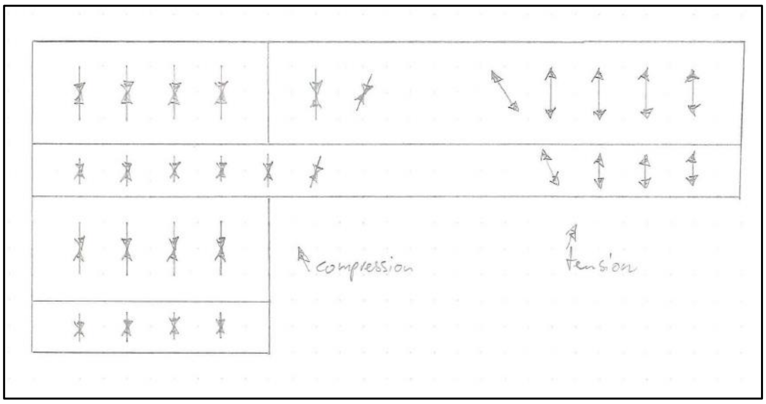


Figure 4.25: Compression and tension in the element

The influence of not providing a proper bond between longitudinal and transverse layers in the flange is investigated in the analysis for the model in use, paragraph 4.3: In use.

By supporting the flanges during pressing the pressure is distributed more evenly over the element. Again, the same goes for the distribution of stress in vertical direction as for the deformation. The partial support influences the distribution close to its location and has almost no influence directly next to it. There is not enough compression to realize a full bond between layers in the flange. By fully supporting the flange the pressure is distributed evenly over the element and (governing) peak stresses are avoided. A good bond between longitudinal and transverse layers is realized over the whole width of the element, including the flanges.

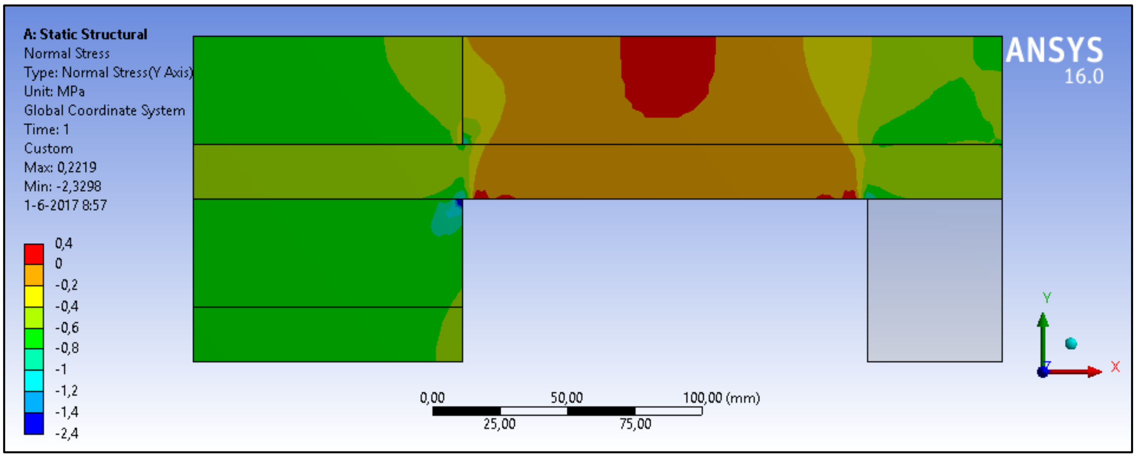


Figure 4.26: Normal stress in y-direction, partially supported

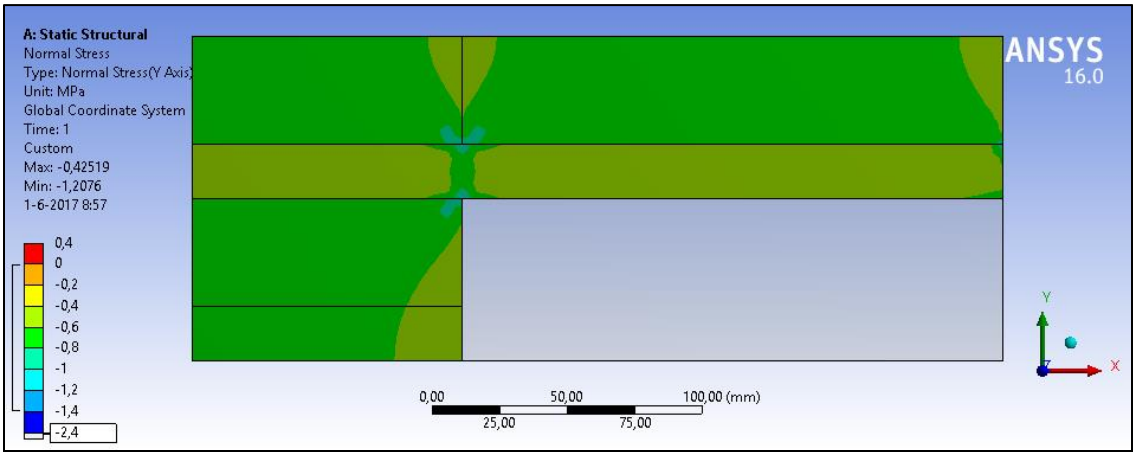


Figure 4.27: Normal stress in y-direction, fully supported

The distribution of normal stress in x-direction shows clear tensile and compressive stresses in the cross layers. This is expected since the cross layer is spanning the hollow core in x-direction. The distribution of these stresses is influenced in the R-profile due to the force reactions as a result of the frictionless supports. The frictionless supports provide the boundary conditions for symmetry and are necessary in the model. In the analysis of the R-profile only compressive forces are observed. The stresses are not governing and do not significantly influence the stress distribution in y-direction, which determines the successful realization of a proper bond.

4.2.5.4 Shear stress

Shear stresses in the xy-plane for both the model without support as for the one with a partial support structure are too high. For the boards oriented in longitudinal direction this is rolling shear. The value for rolling shear is exceeded at the corners of the hollow cores, though the stress only protrudes a couple of mm into the material.

When the full width of the flange is supported during pressing the shear stress decreases significantly and does not govern the design anymore.

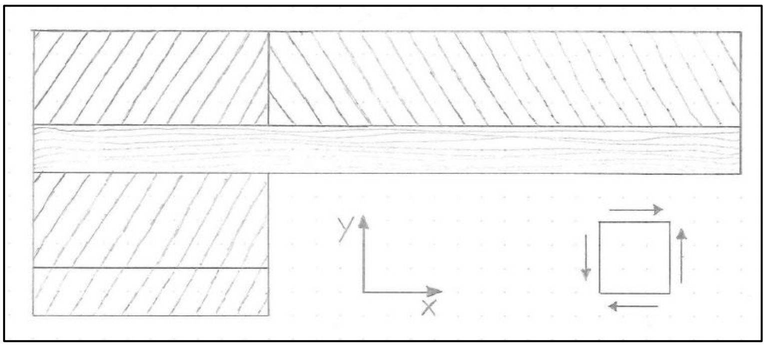


Figure 4.28: Shear stress in the xy-plane

Shear stresses in the yz- and xz-plane do not govern the results.

4.2.6 Conclusion

This paragraph analyses the behaviour during manufacturing. Two models with a different geometry are subjected to a forced deformation, mimicking the pressing procedure, to investigate the behaviour of a HCCLT element.

The smaller R-profile provides more freedom in mesh size and connection options. Small differences in the solution show that the finer mesh does increase accuracy, but this does not significantly alter the final results. The finer mesh does also increase peak stresses at certain points in the element, as long as these peak stresses are not that high and occupy very small areas the stresses are assumed to redistribute.

During pressing the flange shows little resistance to deformation. It can be observed that the deformation in the flange is similar over most of its width, it only differs very close to the web. Here the web and part of the transverse layer provide support and resistance to the deformation. The same holds for the configuration with a partial support, a large part of the flange is still unsupported and "free" to move. Since the flange deforms so easily when not or only partially supported the distribution of normal stress in y-direction shows almost no stress in the flange. This indicates that the bond between longitudinal and transverse layers will not be adequate. As the glue between layers cannot establish a proper bond without pressure.

A full support results in an even deformation over the whole element, with only small discontinuities located at the narrow faces of individual boards. It also shows an even distribution of stress. Note that the support that is used during pressing should move with the element. Saying that the support structure should compress in the same manner as the timber located at the web, this can be realized by using the same material for the support structure or something with the same modulus of elasticity. If the support structure is to rigid, not enough of the pressure will pass through the web and the timber layers in the web will not be bonded properly. On the other hand, when the support structure is not rigid enough the flanges will not bond properly.

Adjustments can be made to automate the process. But, the configuration and dimensions of the hollow cores do influence the manufacturing process. Support structures are necessary to distribute stresses evenly in order to produce a proper bond.

4.3 IN USE

Cross-Laminated Timber is a relatively new product which has not been optimized for the wide area of applications it is being used for. The solid timber elements can be best compared to heavy construction systems, with a large cross sectional area. The design is often determined by serviceability criteria, such as deformation and vibration. The effective bending stiffness and stress distribution depend largely on the rolling shear modulus of the cross layers. This causes a larger deformation due to shear than for other wood based products. Strength criteria, like bending and shear are mostly not governing.

The design methods researched in chapter 3 show little difference in load bearing capacity between standard CLT and the configuration with hollow cores. The configuration of a HCCLT element provides a possibility of reducing the cross sectional area. According to the design methods the design is still governed by the deformation.

Two models are made and analysed in this paragraph to determine the behaviour of HCCLT. The results are compared to those found with the design methods.

4.3.1 General information

Table 4.1: Material properties C24

Density	ρ_{mean}	420	[kg/m^3]
Tensile strength	$f_{t;0;k}$	14	[N/mm^2]
	$f_{t;90;k}$	0,4	[N/mm^2]
Compression strength	$f_{c;0;k}$	21	[N/mm^2]
	$f_{c;90;k}$	2,5	[N/mm^2]
Shear strength	$f_{v;k}$	2,7	[N/mm^2]
	$f_{r;k}$	1	[N/mm^2]
Modulus of elasticity	$E_{0;mean}$	11000	[N/mm^2]
	$E_{90;mean}$	370	[N/mm^2]
Shear modulus	G_{mean}	690	[N/mm^2]
	$G_{r;mean}$	50	[N/mm^2]

The overall dimensions and thickness of the individual layers can be found in the table below.

Table 4.2: Build-up of the configuration

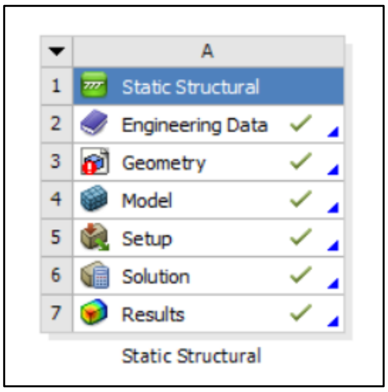
Length span [mm]	7200
Thickness layer 1 [mm]	40
Thickness layer 2 [mm]	20
Thickness layer 3 [mm]	40
Thickness layer 4 [mm]	40
Thickness layer 5 [mm]	40
Thickness layer 6 [mm]	20
Thickness layer 7 [mm]	40
Total height [mm]	240
Width hollow core [mm]	400
Height hollow core [mm]	120

The loading situation is listed below, see paragraph 3.5.6. for a more elaborate specification.

Table 4.3: Loading situation

Permanent loading			
Self-weight	g_e	0,67	[kN/m^2]
Screed	g	1,00	[kN/m^2]
Variable loading			
For an office area	q	2,00	[kN/m^2]
Movable partitions	q	0,80	[kN/m^2]

4.3.2 Engineering data



The material library of ANSYS does not include timber properties. The engineering data used for the analysis can be found in Table 3.1. Since timber is anisotropic, two new materials are added to the library, Timber C24 and Timber C24 Transverse (the latter is used for the cross layers).

4.3.3 Geometry

The models made in Autodesk Inventor are imported into the geometry of ANSYS for analysis. To analyse the behaviour of a HCCLT element in use, two models are made. An II-profile with a length of 7,2 m and an C-profile with a length of 3,6 m. The C-profile makes use of boundary conditions to insure the symmetric conditions. By limiting the length and overall size, the model of the C-profile provides more freedom in choosing mesh size, contact options, etc. The results of the C-profile are used to analyse the behaviour of a HCCLT element, the outcome of the II-profile is used to check these results.

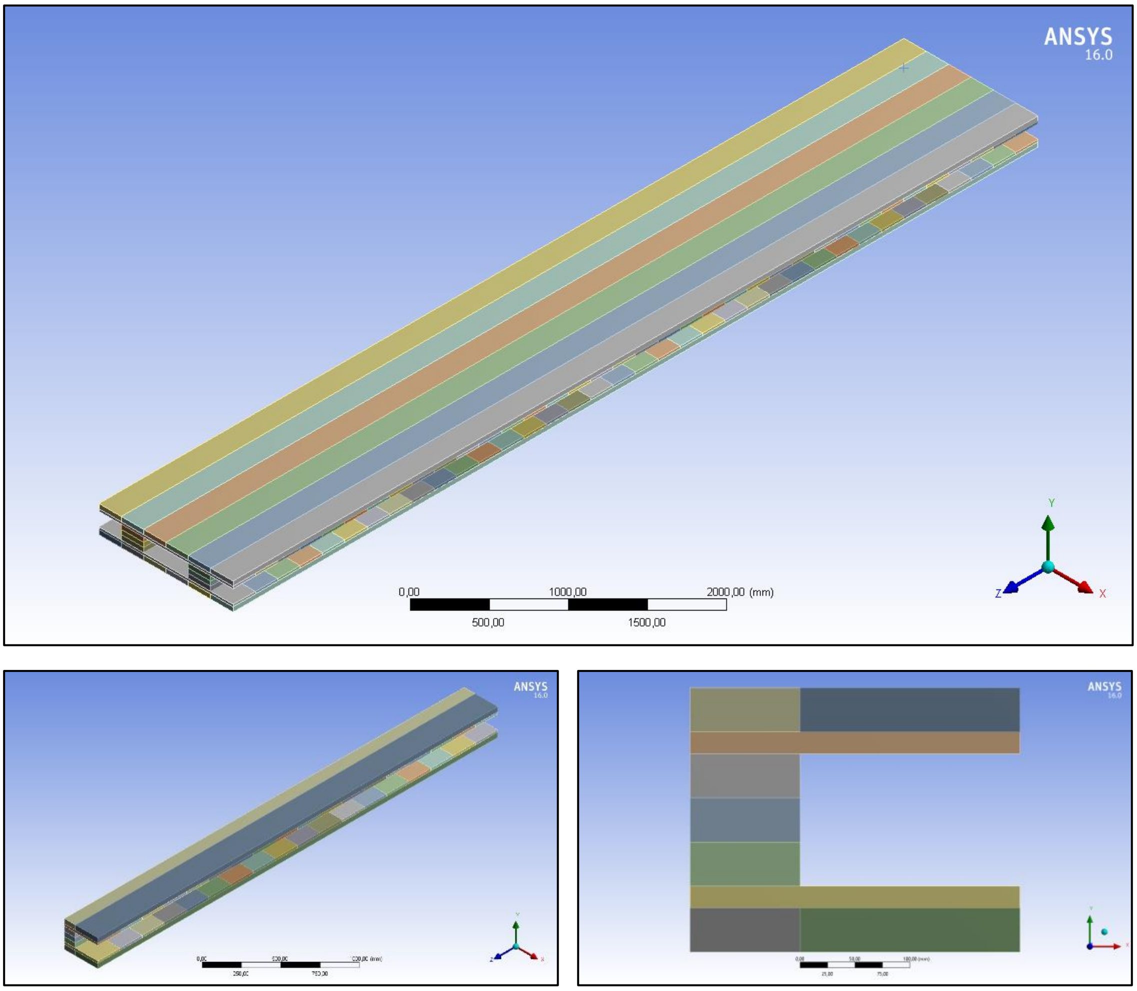


Figure 4.29: II profile of 7,2 m and the C profile with a length of 3,6 m

4.3.4 Model

4.3.4.1 Connections

In the analysis performed to investigate the behaviour during pressing it is clear that without any support structure located in the hollow cores the layers do not bond properly in the flanges. In order to determine the influence this has on the element the connection options for the models are formulated with and without proper bond in the flanges of the element.

Pressing the element with the span over the hollow core fully supported results in a proper bond over the whole element. The contact regions for the model with proper bond in the flanges are separated into two categories. Connections between the wide faces and narrow faces of the individual boards, the following contact options are used. Since there is no edge bond, the side by side contact between boards is suppressed. Compared to using the frictionless contact option the computing time and power needed are much lower and still provide accurate results provided the following holts: It should be checked that the elements representing the individual boards do not significantly crossover / overlap, since this is not possible and may influence the outcome of the analysis. All contacts on the wide faces are modelled as bonded.

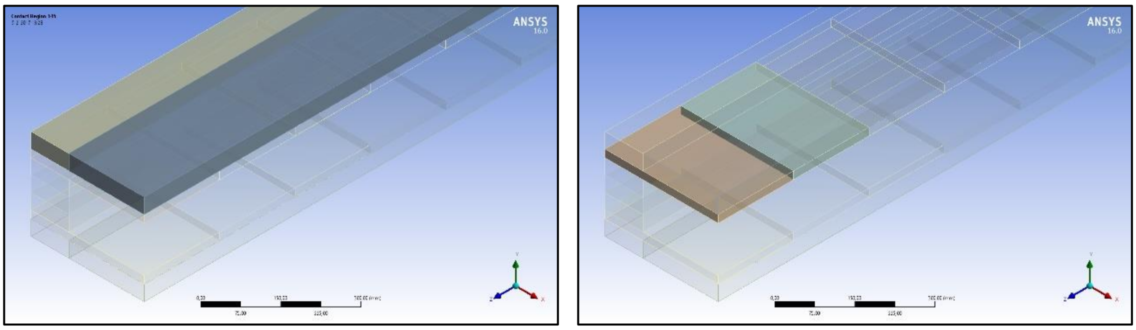


Figure 4.30: Contact regions on the narrow faces

When the pressing of the element does not result in an adequate bond between the layers above the hollow cores, this should be accounted for in the analysis. Meaning the connection may not be modelled as bonded. Suppressing this connection is not an option, because the layers will be pressed through each other, which is not possible. These connections should therefore be set to frictionless or no separation in order to get reliable results.

Because of the large number of contact regions no longer modelled as bonded the overlap between elements is significant. In order to model the behaviour the contact option no separation is used for all contact regions on the narrow faces and for the wide faces located in the flanges. This provides an accurate representation of reality. Some movement is possible as long as the individual boards can be expected to not separate under the specific load case.

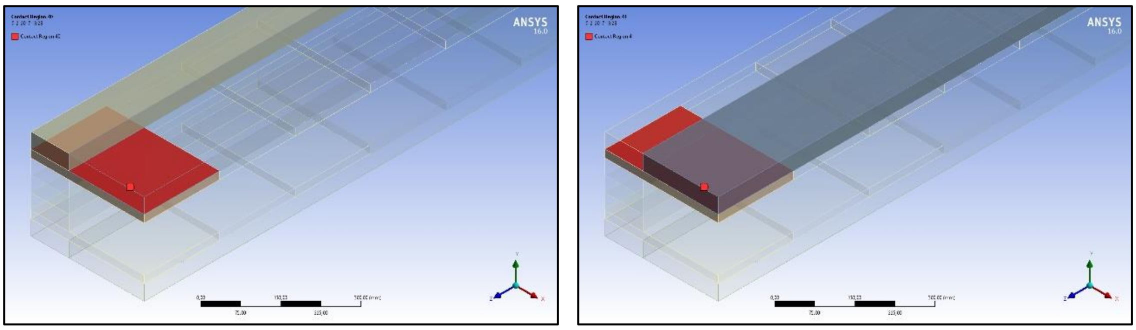


Figure 4.31: Contact regions on the wide faces

4.3.4.2 Mesh

Many different mesh shapes and sizes can be chosen to perform the analysis. And these can be further refined at specific locations, edges etc. The following mesh is used to provide accurate results and maintaining an even distribution over the whole element, which in turn also contributes to the accuracy of the results. [39]

For both the II-profile and C-profile a square mesh is taken. This shape best suits the situation, because of its even distribution and symmetry over the model. Limits for the number of elements determined sizing. Resulting in a 20^3 mm^3 mesh for the II-profile and a 10^3 mm^3 mesh for the C-profile.

Due to the size of the element a finer mesh is not possible and seen the little differences compared to a mesh of 20x20x20 mm^3 it is unlikely it would result in more accurate results. Peak stresses will definitely increase, however deflection and rough stress distribution will not be impacted as much. The mesh is limited to two mesh-elements over the height of the cross layer, due to the chosen mesh distribution. However, an even and symmetrical distribution of the mesh over the whole element will give the most accurate and reliable results, also when this means that the cross layer only has two mesh-elements over its height.

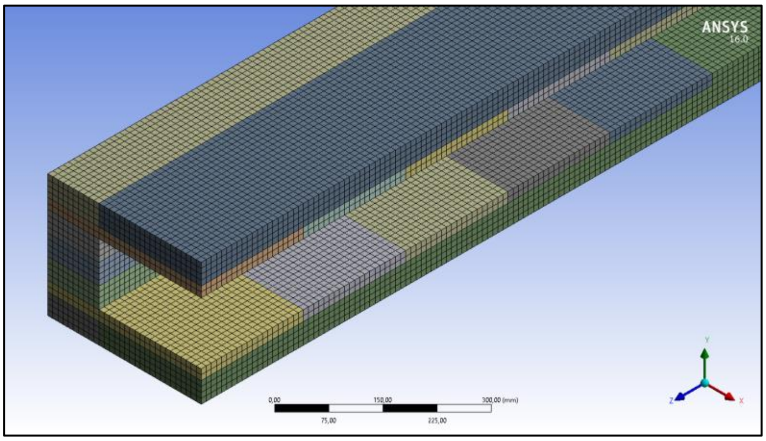


Figure 4.32: Mesh distribution

4.3.5 Setup

To determine the behaviour of a HCCLT element in use, analysis are performed on a simply supported configuration with a span of 7,2 m. The C-profile is modelled from the right support to the middle of the span, where symmetry conditions are applied.

4.3.5.1 Displacements & frictionless supports

The support structure is modelled with displacement restrictions and additional frictionless supports for the C-profile, which provide the boundary conditions for the symmetry.

For the II-profile both ends are supported in y-direction, where the displacement restrictions are set to zero. Only the displacement is restricted not the rotation. Note that the support structure in practise should provide sufficient surface area to distribute stresses. Vertices on the sides of the model are restricted from moving in either x- or z-direction, see Figure 4.33.

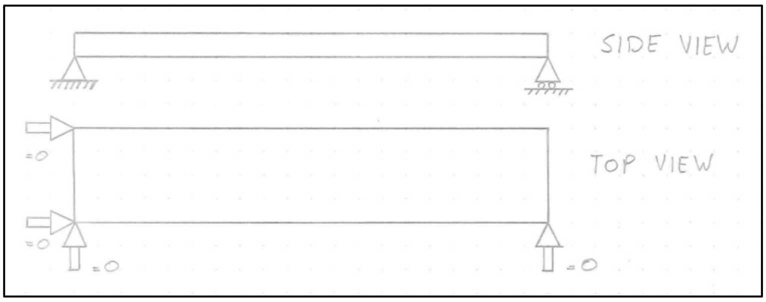


Figure 4.33: Displacement restrictions II-profile

The C-profile uses the same support, displacement restriction in y-direction, on one end and makes use of a frictionless support on the other. The frictionless support is applied on the left end, which would be in the middle of the 7,2 m span. Restrictions for moving in x- and z-direction are enforced by the frictionless supports used for the symmetry of the model. All faces located in the middle of the web are supported by a frictionless support and for the faces located at the hollow cores only those of the cross-layers are supported. This is done because these are the only ones that actually got cut-off, the boards in longitudinal direction end precisely in the middle of the cross-span and there is no edge bond present.

The frictionless supports of the C-profile can be seen in Figure 4.34, only boards that are cut-off are supported.

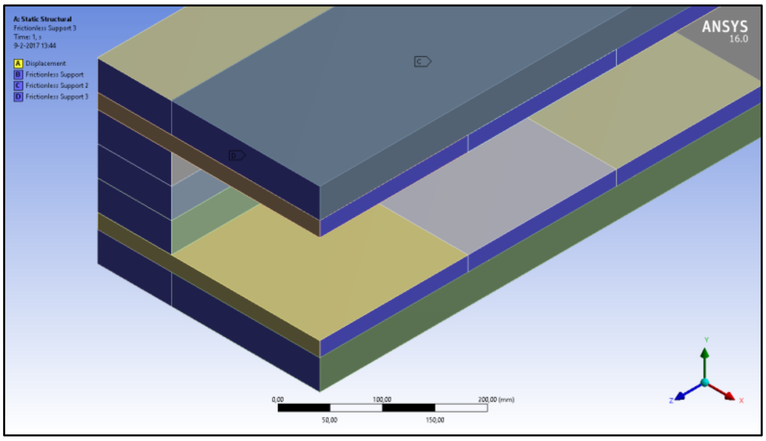


Figure 4.34: Frictionless supports C-profile

4.3.5.2 Standard earth gravity & pressure

The loading depends on the weight of the element, floor covering and the live load. The density of the timber is known and supplied into the engineering data. Standard earth gravity is then used to introduce the self-weight onto the element. The floor covering and live load are modelled with a pressure applied on the top face of the element.

The standard earth gravity and pressure, that are applied to the model, can be seen in Figure 4.35.

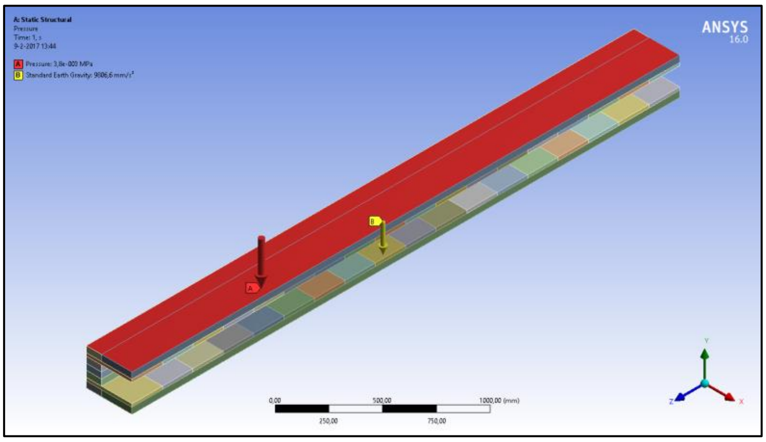


Figure 4.35: Loading situation

4.3.6 Solution and results

The models are simply supported and loaded by a uniformly distributed load. The shear and moment distribution for such a configuration can be seen in Figure 4.36. Maximum deformation and normal stresses will occur in the middle of the span, for the model of the C-profile this is the left end. The Shear stress is maximum at the supports.

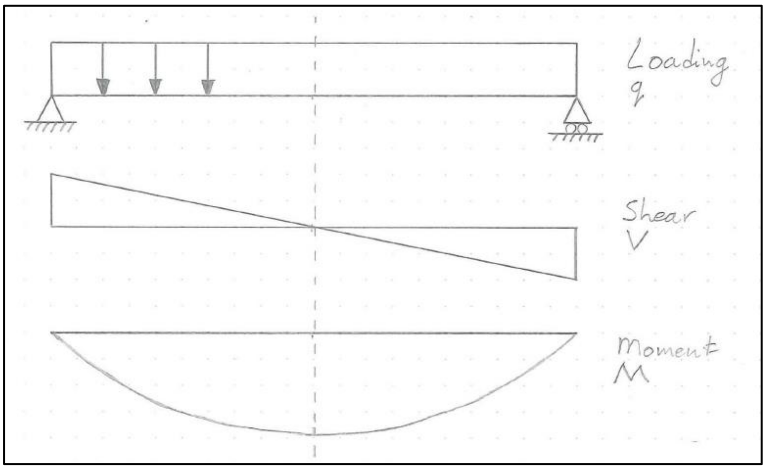


Figure 4.36: Shear & moment distribution, simply supported structure under uniformly loading condition

4.3.6.1 Deformation

The deformation of the C-profile can be seen below. This is the deformation for the model with proper bond in the flanges over the hollow cores. Maximum deformation occurs on the left end of the model, this is where the model is supported by frictionless supports, located in the middle of the 7,2 m span.

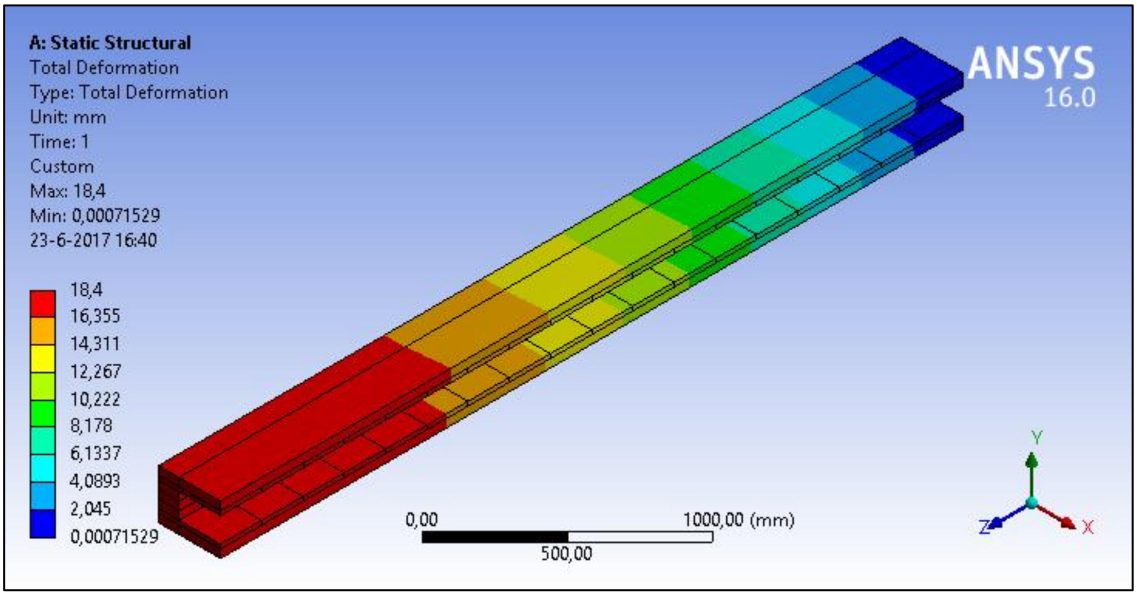


Figure 4.37: Deformation

The deformation in the middle of the span is 18,4 mm. This is the instantaneous deformation. Additional factors for creep and time further determine the final deformation.

In paragraph 4.2 where the behaviour during manufacturing is investigated, it was found that without taking any measures for the pressing procedure the bond in the flanges would not be adequate. To see what the effect of this would be, a model was analysed with contact options as described in paragraph 4.3.4. This resulted in a deformation almost twice as high. Showing that when there is no bond present in the flanges the "loose" boards contribute little to the capacity of the element.

4.3.6.2 Normal stress

The model is spanning in z-direction and as a result of this the maximum normal stresses can be found in this direction. The maximum normal stress occurs where the moment is largest, for a simply supported configuration this is in the middle of the span. Figure 4.38 shows the normal stress distribution for the C-profile.

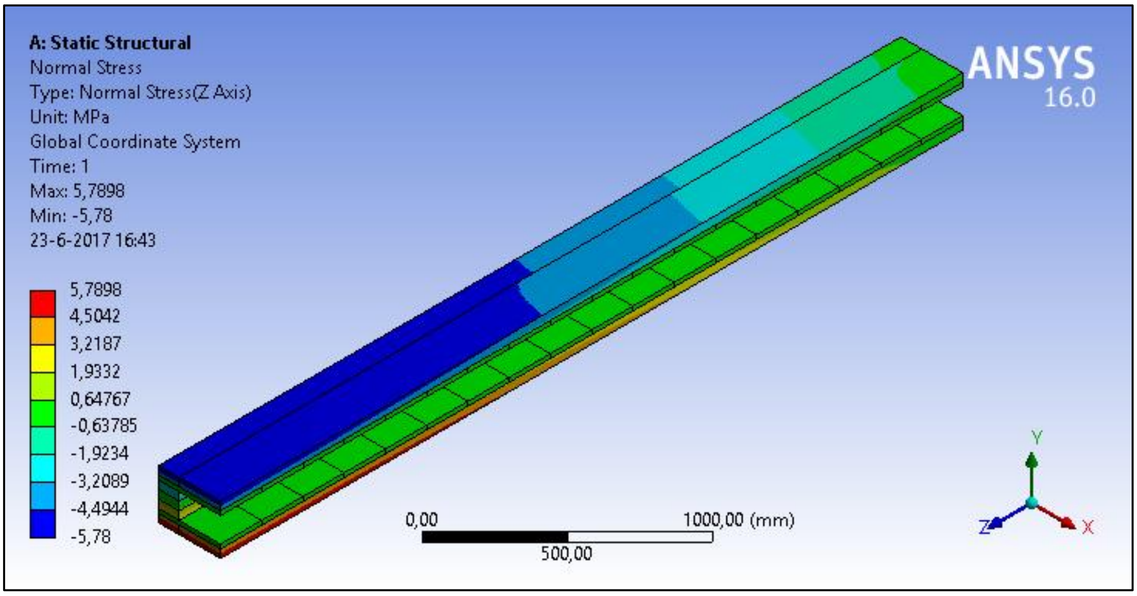


Figure 4.38: Normal stress distribution in z-direction

In Figure 4.39, the cross section halfway the span is shown. There is a small discontinuity in normal stress between boards in the top and bottom layers. This discontinuity is due to the fact that there is no edge bond present between boards. The contribution of the boards located in the flange is slightly less than that of the boards located directly above the web.

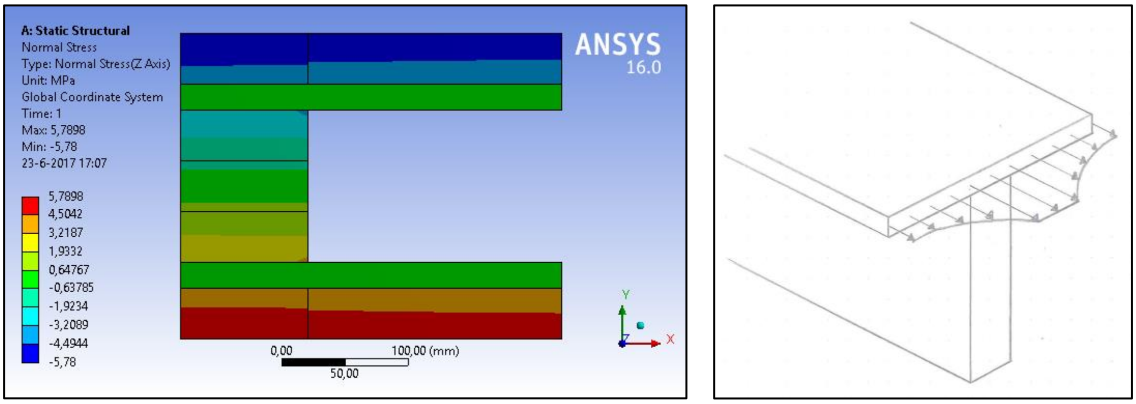


Figure 4.39: Normal stress distribution in z-direction and the effective width of the flange

Due to shear deformations, the normal stresses in the flanges are not uniformly distributed.

The contribution of the boards, oriented in longitudinal direction and located in the flange, can be defined as a percentage that translates in the effective width of the flange. In the middle of the span the effective width is 96,5 %. Going from the middle of the span closer to the support the effective width slightly decreases, showing an effective width of more than 95 % at a quarter of the span.

The normal stress distribution in z-direction shows individually stressed boards in the cross-layers. This indicates that suppressing the contact regions on the narrow faces of the boards has the desired effect, resulting in no edge bond.

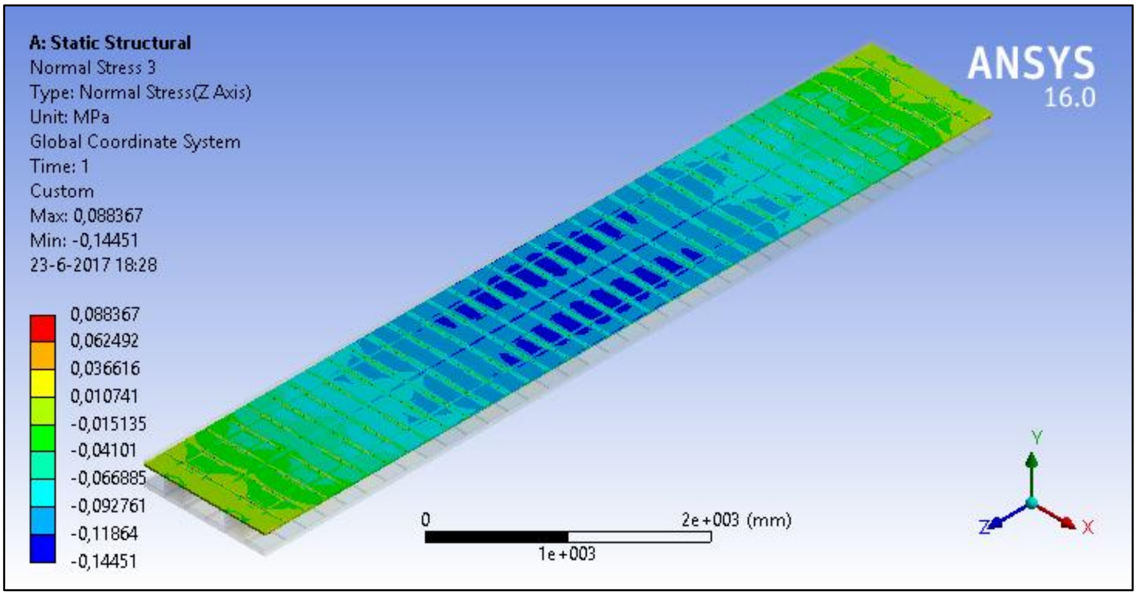


Figure 4.40: Normal stress distribution in z-direction for the top cross layer

The transverse layers, spanning the hollow cores, are stressed in x-direction. Figure 4.41 shows the distribution for the II-profile. The distribution of stress in the C-profile is somewhat influenced by the frictionless supports. The boards in the middle of the model show a distribution expected to see for a board spanning over two supports and loaded in y-direction. The boards closer to the support show higher stresses and an influence by the shear stress.

There is also a difference between the cross-layers near the top of the element and those near the bottom. Since the cross-layers near the top are almost directly subjected to the pressure applied on top of the model, they show a larger deflection and higher normal stresses.

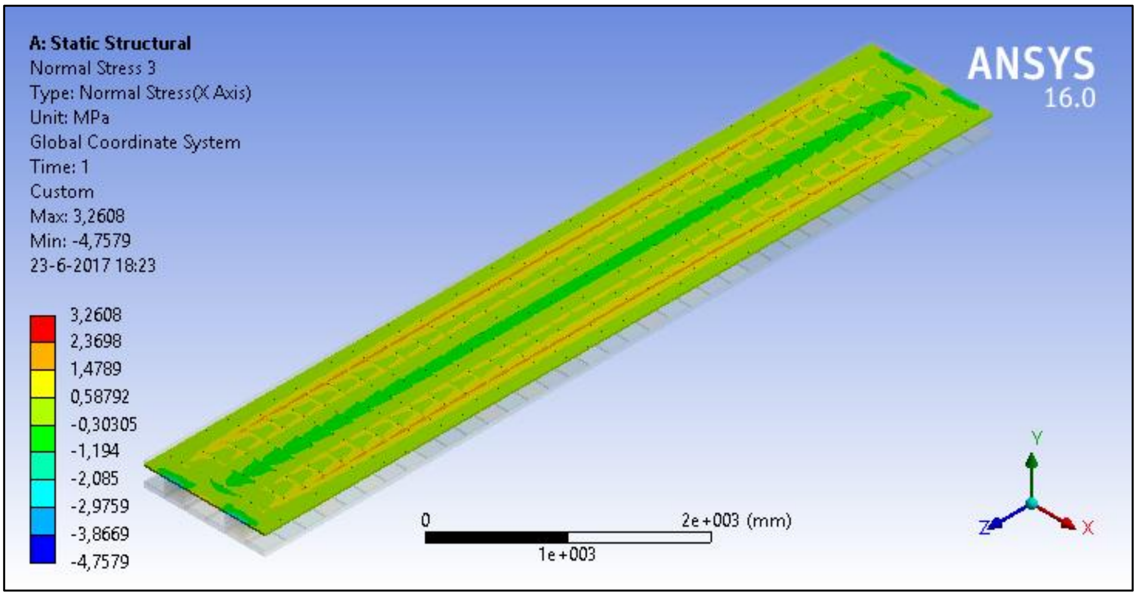


Figure 4.41: Normal stress distribution in x-direction for the top cross layer

4.3.6.3 Shear stress

The shear stress found for a simply supported span loaded with a uniformly load is largest at the supports. The shear distribution shown in Figure 4.36 can be compared to the shear distribution found in the yz-plane for the model in ANSYS.

The cross-layers transfer the stress from the outer layers, including flanges, to the web. This is done through rolling shear, for the shear in the yz-plane, see Figure 4.42.

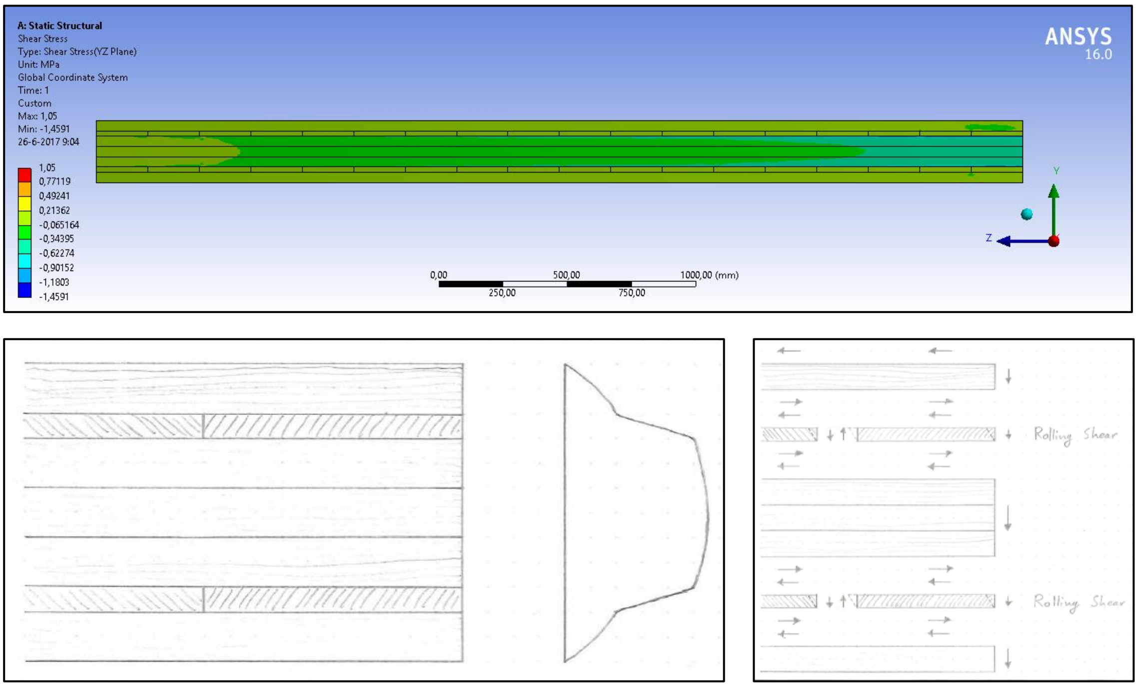


Figure 4.42: Shear stress distribution in the yz-plane

A similar shear distribution, as shown in Figure 4.43, can be found for an I-beam, which could be expected. Small peak stresses are located at the corners of the hollow cores, but are still within the limits. The average distribution is similar to that found with the calculations for the design methods, the main differences are the peak stresses due to the 3D configuration and the fine mesh, used to perform the analysis.

The maximum shear, excluding some small peak stresses, occur in the middle layer. Values of approximately 0,4 N/mm^2 are found. The shear stress in the cross-layers, which are stressed in rolling shear, are approximately 0,2 N/mm^2, with a peak reaching 0,45 N/mm^2.

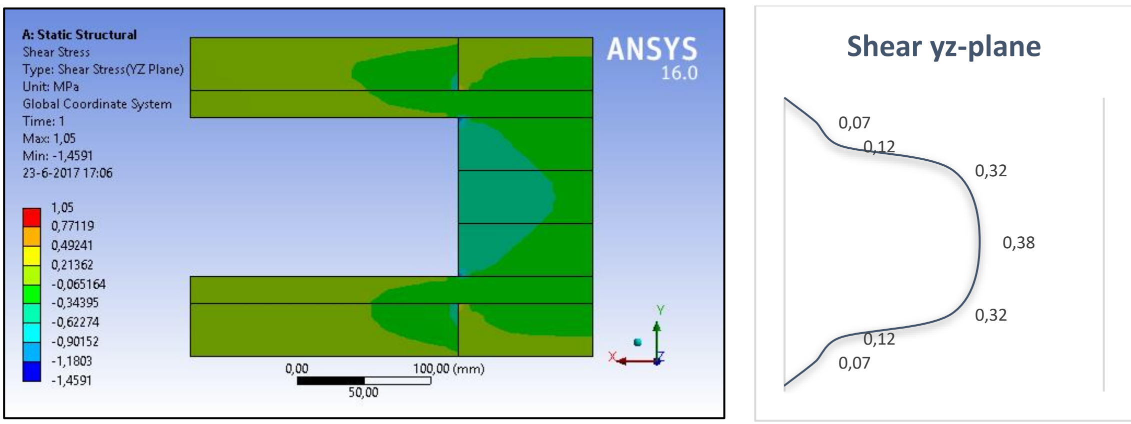


Figure 4.43: Shear stress distribution in the yz-plane

As mentioned in paragraph 4.3.6.2, the normal stresses in the flanges are not uniformly distributed, due to the shear deformations.

The contribution of the boards, oriented in longitudinal direction and located in the flange, can be defined as a percentage that translates in the effective width of the flange. In the middle of the span the effective width is 96,5 %.

The boards are stressed in shear in the xz-plane and this causes torsion between the boards in longitudinal and transverse direction located in the flanges. The difference in stress between the surfaces of the boards result in the 3,5 % loss in capacity.

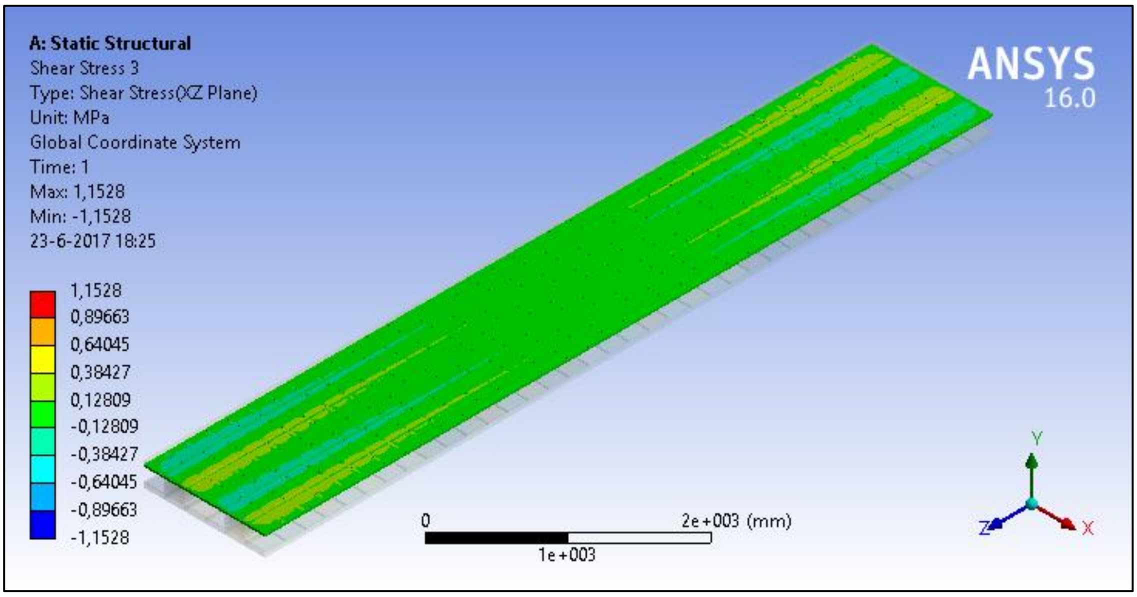


Figure 4.44: Shear stress distribution in the xz-plane for the top cross-layer

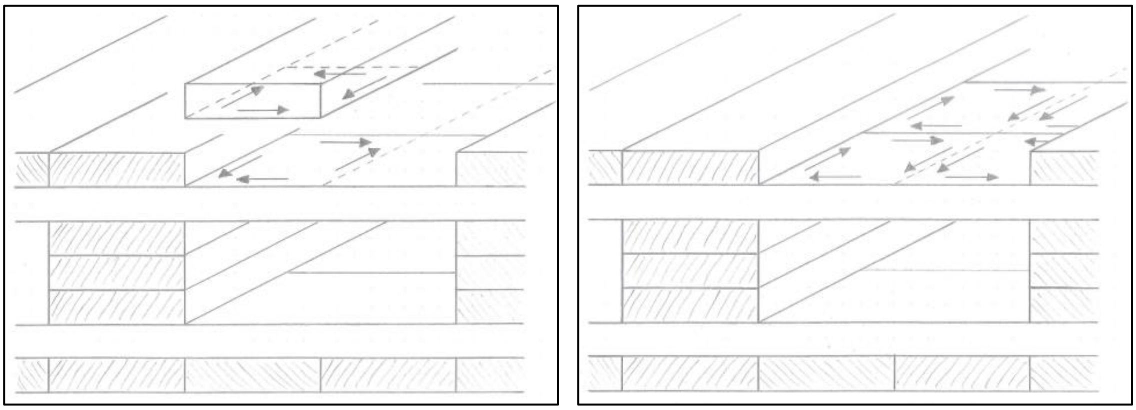


Figure 4.45: Shear resulting in torsion between boards in the flanges

The cross layer is stressed in the xy-plane under longitudinal shear. Although timber has a higher tolerance for longitudinal shear when compared to rolling shear, in the design of HCCLT the cross layer needs to transfer stress from the flange over into the web and this is done through longitudinal shear.

The maximum shear, excluding some peak stresses, occur in the cross-layers. Values of approximately 0,9 N/mm^2 are found. The shear stress in the longitudinal-layers, which are stressed in rolling shear, are approximately 0,1 N/mm^2.

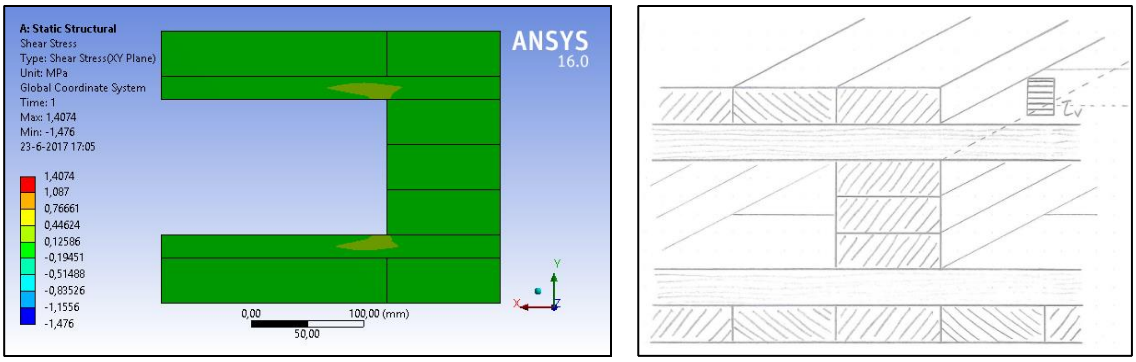


Figure 4.46: Shear stress distribution in the xy-plane

4.3.6.4 Modal analysis

The first two modal shapes result in frequencies of 8,6 Hz and 24,3 Hz. The fundamental frequency derived with the simple calculation from the EC 5 is 6,9 Hz.

The design tables from Derix include designs for floors with frequencies of 6 Hz and 8 Hz. This is close to that found in the analysis. These frequencies, according to the national annex to the EC 0, cannot be induced by people walking or dancing and are assumed not governing the design.

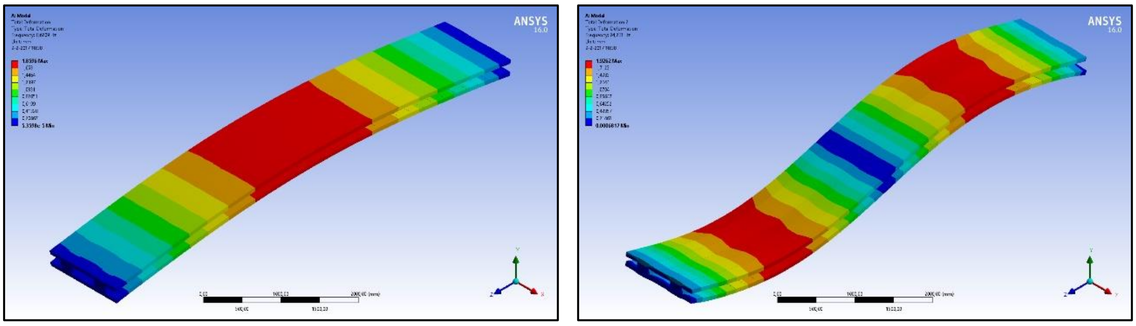


Figure 4.47: First two fundamental frequencies and modal shapes

4.3.7 Comparison

When comparing the results obtained with the design methods in chapter 3, the results are very similar. Again the deformation is governing, with the model showing 18,4 mm compared to 18,1 mm for the instantaneous deformation derived with the calculations.

The stress distribution is somewhat different due to the 3D model, but looking at the mean distribution similar results are observed. The mean normal stress distribution is sketch in Figure 4.48, and comes closest to the distributions found with the shear analogy. The maximum normal stress is 5,7 N/mm^2, which is very close to the 5,8 N/mm^2 found in the calculations.

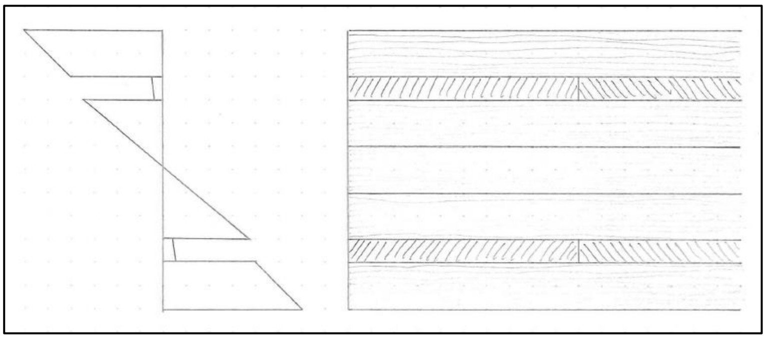


Figure 4.48: Normal stress

The distribution of shear stress is similar to that found with the calculations for the mechanically jointed beams theory and the shear analogy. The results from the model analysed in ANSYS does show some peak stresses located at the edges of individual boards, however the overall stress distribution shows a maximum stress of less than 0,4 N/mm^2 compared to 0,37 N/mm^2, again very similar. The shear stress due to the cooperating flanges in longitudinal direction shows a slightly larger difference. The cross layers are stressed by the transfer of normal force in longitudinal direction from the flange to the web. The model shows a maximum stress of 0,9 N/mm^2 while the derived stress in chapter 3 is 1,075 N/mm^2.

4.3.8 Conclusion

This paragraph analysed the behaviour of a HCCLT element in use. The analysis performed to determine the behaviour during pressing showed that not supporting the structure results in no or an inadequate bond between layers in the flanges. The solutions in this paragraph show that not providing a proper bond in the flanges is not an option, it results in deformations and stresses that become much higher. The "loose" boards contribute very little.

The mesh size is limited by the overall size of the model. Comparing the results from the two models, with different mesh sizes, shows no significant difference in deformation or stress distribution. The results of the C-profile are slightly more accurate and the peak stresses become higher, but the final result is not impacted.

The results obtained in this paragraph are similar to those derived with the calculations in chapter 3. The deformation is still governing the design but not exceeding the maximum allowed deformation. While the normal and shear stresses show a similar distribution with somewhat lower values.

5 CONCLUSIONS AND RECOMMENDATIONS

Answers to the research questions of this thesis will be presented in this chapter and recommendations are given for further study or to provide additional inside.

What advantages can be realized with Hollow Core Cross-Laminated Timber systems?

> *What are the consequences of the manufacturing process on the configuration and dimensions of the hollow cores?*
>
> *What effect will the hollow cores have on the structural properties of an element?*
>
> *What effect will different dimensions of the hollow cores have on the economical use of material?*

5.1 CONCLUSIONS

- The mechanical behaviour of a HCCLT element during the manufacturing process, as described in chapter 2 and paragraph 4.2, shows it is not possible to press an element without taking additional measures. The normal stress is not evenly distributed over the element and there is no adequate bond realized at the location of the flanges. High shear stresses located at the corners of the hollow cores further add to the problem.

  The configuration and dimensions of the hollow cores is not limited by the manufacturing process. The maximum size of an element is still 17,8 m in length and 3,5 m in width, with a thickness of 400 mm. However, the hollow cores do have an impact on the manufacturing process, since support structures are necessary during the pressing process.

- There are a variety of different methods used to design CLT structures. The three most used are researched and described in order to then be used to calculate three different configurations. One of these configurations concerns a HCCLT element.

  The results obtained with the design methods and the analysis performed in ANSYS show very similar results. In most cases the ANSYS models shows slightly lower values, making the design methods a little more conservative. The conclusion based on this work is that the design methods are a good starting point for the design of CLT as well as for HCCLT.

While the composite method does not account for shear deformation and both the mechanically jointed beams theory and the shear analogy do, the outcome shows little difference, concerning the stresses, deformation, etc. With the mechanically jointed beams theory from the EC 5 as the most conservative, showing the lowest effective bending stiffness. In the case of these specific configurations the choice of design method does not significantly influence the result.

When comparing HCCLT to the traditional configuration of CLT the effective bending stiffness shows no significant influence by the reduced cross-sectional area. The HCCLT configuration does show larger values for shear. Because of the hollow cores there is less material in the middle of the element, resulting in shear stresses between 2 to 3 times higher compared to the other configurations. Still the values are well within the limits.

An extra shear check is performed on the HCCLT configuration, for the configuration calculated in chapter 3 this is not governing the design. However, the values are such that it may influence the thickness of the cross-layers for other specific cases and should be looked into.

The already small contribution of the dead weight to the loading is not altered much by the reduced net cross-sectional area of the HCCLT configuration and as a result does not significantly affect the fundamental frequency.

- Optimisation of material usage is possible.

  The possible size of the hollow cores depends on the length of the span, loading situation and element configuration. In the case of examples two and three from chapter 3 a reduction of more than 30% for the net cross-sectional area is realised with the hollow cores. The effective bending stiffness is almost the same, showing a much more efficient use of material.

5.2 RECOMMENDATIONS

- To produce HCCLT by pressing it in its entirety an support structure is needed during pressing. Based on this work the whole flange needs to be supported. Increasing the thickness of the cross-layers showed a slight improvement in stress distribution. When a standard configuration is determined, regarding the length of the span, loading situation, etc. the cross-layer thickness, support structure and possibly other additional influential factors should be optimized.

  Some manufacturers use vacuum presses. It is recommended to investigate the possibilities of this pressing process for the production of HCCLT. The theoretical maximum pressure is significantly lower and this could have an impact on the structural properties. However, when part of the mould, airbag or tarp could be placed inside of the hollow cores, the pressure should be equal over the whole element. And after the pressure is relieved easy to remove.

- The HCCLT configuration analysed in this thesis shows almost the whole flange contributing to the capacity of the element. And the effective width is further improved when an edge bond is realized between boards in the outer layers. When it is favourable to increase the size of the hollow cores this should be taken into account for optimizing the configuration.

  The cross-layer is loaded in shear in order to transfer the stress from the outer layers, including those in the flanges, to the web. Making the cross-layer thicker slightly improves the distribution of stress during the manufacturing process. However, a thicker cross-layer influences the behaviour of the element. The gamma factor used in the mechanically jointed beams theory is partially affected by the thickness of the cross-layer. This has to be taken into consideration.

- It would be highly recommended to calculate and test multiple configurations. From the results in this work it appears that the calculation methods are close to reality. Note that this is one configuration of infinitely many, with a relatively large span over depth ratio.

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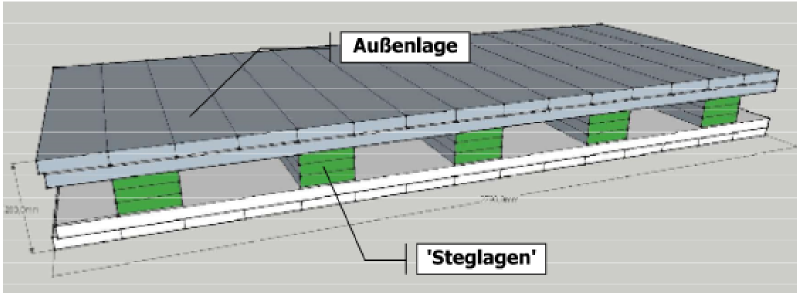
APPENDICES

A ABMESSUNGEN UND AUFBAU VON X-LAM-STEGPLATTEN

Abmessungen und Aufbau von X-Lam-Stegplatten

Eigenschaft	Wert

Systemskizze:



Beschreibung des Bauprodukts:
Derix X-Lam-Stegplatten werden prinzipiell wie Brettsperrholz hergestellt. Der Querschnitt der Stegplatte ist symmetrisch bezogen auf die Mittellage. Die einzelnen Lagen bestehen aus nach Festigkeit sortierten, keilgezinkten und faserparallelen Brettern. Die 'Stege' bestehen aus maximal sechs faserparallel verklebten Längslagen. Bei Elementen mit mindestens sieben Lagen dürfen bis zwei benachbarte äußere Lagen faserparallel verklebt sein.

Dicke	120-400 mm
Breite	≤ 3,50 m
Länge	≤ 18,00 m
Anzahl Lagen ....Gesamt ....faserparallele 'Stege' ....faserparallele Außenlagen	3 ≤ n ≤ 11 1 ≤ n ≤ 6 ≤ 2
Maximaler lichter Abstand der Steglagen	≤ 500 mm
Überhöhung in Plattenlängsachse	≤ ℓ/200tel
Bretter	
Material	Fichte, Tanne, Kiefer, Douglasie und Lärche
Holzgüte nach EN 338 bzw. EN 14081-1	≥ C16
Dicke .... der Außenlagen .... der Querlagen .... der Steglagen	15 bis 45 mm 30 bis 45 mm 30 bis 45 mm
Breite .... der Plattenlagen .... der Stege	120 bis 260 mm 160 bis 260 mm
Holzfeuchte nach EN 13183-2	8±2; 9±2; 10±2; 11±2; 12±2 (in %) Innerhalb eines Brettsperrholzplattenelements darf nur eine der angegebenen Feuchtigkeitsbereiche angesetzt werden
Keilzinkenverbindung	Nach EN 385

B PRODUCT INFORMATION GRIPPRO™ PLUS

AkzoNobel Wood Coatings
Wood Finishes and Adhesives

Product Information

Plus

AkzoNobel System GripPro™ Plus besteht aus Plus A011, einem flexiblen, flüssigen Melamin Leim und Plus H011, einem flüssigen Härter.
Es handelt sich um ein helles Leimsystem, welches in gemischtem oder getrenntem Auftrag von Leim und Härter für tragende Holzkonstruktionen, wie z. B. Brettschichtholz, Brettsperrholz oder Duo-/Trio-Balken eingesetzt werden kann.

Plus wird in der holzverarbeitenden Industrie eingesetzt, wo Anforderungen an eine helle Leimfuge mit hoher Wasser- und Wetterfestigkeit gestellt werden.

Plus wurde gemäß den Anforderungen in EN 301:2013 als Klebstofftyp I durch die Materialprüfungsanstalt Universität Stuttgart – Otto-Graf-Institut - (MPA), Deutschland, für ein flexibles Mischungsverhältniss geprüft und anerkannt (siehe unten). Das Produkt ist für die Herstellung von Brettschichtholz gemäß EN14080:2013 geeignet.

Das Klebstoffsystem erfüllt die Anforderungen folgender Typen:

EN 301-I-90-GP-0,6-M
EN 301-I-90-GP-0,3-S
EN 301-I-90-FJ-0,1-M

Des Weiteren ist das Leimsystem durch die Materialprüfungsanstalt Universität Stuttgart – Otto-Graf-Institut - (MPA), Deutschland nach DIN 68141:2008, geprüft worden und erfüllt die Anforderungen an die Produktion von tragenden geklebten Holzbauteilen gemäß DIN 1052 für ein flexibles Mischungsverhältniss (siehe unten).

Bei getrenntem Auftrag von Leim und Härter wird der Einsatz der Gießanlage 6230 oder 7230 Ecoflex empfohlen.

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AkzoNobel approval code: AN_200100_210114

Version: 4 (2015-06-15)

Reason for changes: update according to EN14080:2013

Produktspezifikation

	Plus A011	Plus H011
Produkt	Melamin Klebstoff	Härter
Lieferform	Flüssig	Flüssig
Farbe	opak weiß	Weiß
Viskosität *(zum Zeitpunkt der Produktion)*	2000 - 9000 mPas (Brookfield LVT, sp.4, 12 rpm, 25ºC)	1700 - 2700 mPas (Brookfield LVT, sp4, 60 rpm, 25°C)
pH *(zum Zeitpunkt der Produktion)*	8,5-9,3 (bei 25°C)	1,3 – 2,0
Trockengehalt	Ca. 65%	Entfällt
Dichte	ca. 1290 kg/m³	ca. 1070 kg/m³

Lagerbedingungen und Lagerstabilität

Um die angegebene Lagerstabilität zu gewährleisten ist es äußerst wichtig, dass das Produkt unter den empfohlenen Lagerbedingungen bevorratet wird.

Die optimale Lagertemperatur für den Leim liegt zwischen 5°C und 20°C.
Nur kurzfristige Lagerung bei Temperaturen unter +5°C und über +30°C zulässig. Das Produkt darf gefrieren, muss dann jedoch aufgetaut, auf Raumtemperatur gebracht und vor Gebrauch homogenisiert werden.

Die optimale Lagertemperatur für den Härter liegt zwischen 15°C und 25°C.
Nur kurzfristige Lagerung bei Temperaturen unter +10°C und über +30°C zulässig. Gefrorenes Produkt kann, aufgrund irreversibler Veränderungen, nicht wieder aufgetaut und verarbeitet werden.

Die Lagerstabilität eines Produktes wird durch Parameter wie z. B. Reaktivität, Viskosität oder Rheologie bestimmt. Die Lagerfähigkeit endet, sobald sich die Reaktivität, Viskosität oder Rheologie von einem stabilen Wert in einen Wert, der die Verleimqualität beeinträchtigt, umwandelt.

Wenn das Gebinde über einen längeren Zeitraum unverschlossen ist, ist der Leim anfällig für Hautbildung an der Oberfläche. Zur Vermeidung halten Sie die Verpackung stets geschlossen, wenn sie nicht in Gebrauch ist.

Die Lagerzeit der Komponenten finden Sie bitte nachfolgend:

Lagerfähigkeit (Monate)		15°C	20°C	25°C	30°C
	Plus A011	4	3	1,5	1
	Plus H011	4	4	3	2,5

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Verarbeitungshinweise

Plus wurde für die Verwendung in der holzverarbeitenden Industrie in Anwendungsbereichen wie der BSH-Produktion gemäß EN14080:2013, CLT, Duo- und Trio-Balken sowie I-Träger entwickelt.

Mischungsverhältnis

Plus ist gemäß EN301:2013 für nachfolgendes Mischungsverhältnis zugelassen:

Fichte, Kiefer, Tanne (nach Gewichtsteilen)	gemischt bei Keilzinkenverklebung gemischt und getrennt bei Flächenverklebung	100 : 10-100 (Leim : Härter) 100 : 30-100 (Leim : Härter)
Europäische Lärche (nach Gewichtsteilen)	gemischt bei Keilzinkenverklebung gemischt und getrennt bei Flächenverklebung	100 : 30-60 (Leim : Härter) 100 : 30-60 (Leim : Härter)
Sibirische Lärche (nach Gewichtsteilen)	gemischt bei Keilzinkenverklebung gemischt und getrennt bei Flächenverklebung	Prüfung läuft Prüfung läuft

Leim und Härter müssen entsprechend dem oben genannten Mischungsverhältnis verwendet werden. Bei abweichendem Mischungsverhältnis werden unterschiedliche Faktoren, wie z. B. Presszeit, Topfzeit, Wartezeiten sowie die Leimfugenqualität beeinflusst.

Die maximal zulässige Abweichung der Härtermenge beträgt bei der Produktion von tragenden Holzbauteilen ±2 Gewichtsteile.

Vor der Verwendung der Leimmischung im Untermischverfahren muss auf eine sorgfältige Vermischung von Leim und Härter geachtet werden. Bei manueller Vermischung von Leim und Härter immer den Härter dem Leim zuführen.

Getrennter Auftrag von Leim und Härter

Plus ist für den getrennten Auftrag von Leim und Härter zur Flächenverleimung optimal geeignet, vorzugsweise mit der getrennten Gießanlage 6230 oder 7230 Ecoflex. Diese Anlagen gewährleisten eine exakte Dosierung beim Leim- und Härterauftrag. Die maximalen Wartezeiten werden bei gleichzeitiger Beibehaltung der kurzen Presszeiten verlängert.
Der Einsatz anderer getrennter Auftragsgeräte ist nur zulässig, wenn die Eignung der entsprechenden Anlage für die beabsichtigte Anwendung überprüft wurde.
Bei der Verwendung von Leim und Härter im getrennten Verfahren werden keine Probleme mit der Topfzeit auftreten, da die Komponenten erst beim Auftragen auf die Fügeteil-Oberfläche vermischt werden.

Die maximal zulässige Klebfugendicke bei getrennter Anwendung von Leim und Härter bei der Flächenverleimung beträgt 0,3 mm.

Untermischanwendung von Leim und Härter

Plus kann auch als Untermischsystem für Keilzinkenverklebungen verwendet werden, vorzugsweise mit automatischen Mischvorrichtungen. Hierbei ist die Einhaltung der Topfzeit zu beachten, da diese die Verarbeitungsdauer für das Leimsystem einschränkt.
Unter Topfzeit versteht man die Zeit, während der die Leim-/Härtermischung verarbeitet werden kann. Die Topfzeiten werden anhand genormter Analysemethoden gemessen, so dass die Topfzeiten unterschiedlicher Systeme verglichen werden können.

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Nachfolgende Topfzeit wurde gemäß EN302-7 bestimmt:

	Mischungsverhältnis	15°C	20°C	30°C
Topfzeit	100:10	4 h 20 min	1 h 50 min	
	100:30	---	50 min	
	100:100	---	12 min	7,5 min

Wartezeit

Die Wartezeit ist die Zeit vom Moment des Leimauftrages bis zum Aufbringen des Pressdruckes auf die Fügeteile.

Die gesamte Wartezeit setzt sich aus offener (OAT) und geschlossener (CAT) Wartezeit zusammen. OAT ist die Zeit vom Aufbringen des Leims bis zum Zusammenlegen der Fügeteile. CAT ist die Zeit vom Zusammenlegen der Fügeteile bis zum Aufbringen des Pressdruckes.

OAT und CAT werden durch die Leimauftragsmenge, den Feuchtigkeitsgehalt des Holzes und die Raumtemperatur sowie Luftfeuchte beeinflusst. Höherer Leimauftrag, niedrigere Temperatur sowie höherer Feuchtigkeitsgehalt im Holz und in der Luft verlängern die OAT und CAT.

Der Pressdruck muss aufgebracht werden, solange der Leim klebfähig ist.

OAT und CAT -Werte sollten getrennt voneinander betrachtet werden. Die gesamte Wartezeit (OAT + CAT) muss für jeden speziellen Fall bewertet werden. Die offene Wartezeit sollte so kurz als möglich gehalten werden.

Nachfolgende Wartezeiten werden für Plus empfohlen:

	Verhältnis	Leimparameter	Maximale Wartezeit
Wartezeiten, getrenntes Verfahren	100:30	20°C/250 g/m²	1 h
		20°C/400 g/m²	2 h
	100:100	20°C/250 g/m²	35 min
		20°C/400 g/m²	50 min

	Verhältnis	Leimparameter	Maximale Wartezeit
Wartezeiten, Untermisch Verfahren	100:30	20°C/250 g/m²	1 h
		20°C/400 g/m²	1 h 20 min
	100:100	20°C/250 g/m²	25 min
		20°C/400 g/m²	25 min

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Abhängig von der Umgebungstemperatur, der Lamellentemperatur und der Lamellenqualität kann die Leimmenge für spezielle Produktionen optimiert werden. Dieses darf nur in Absprache mit der Anwendungstechnik von AkzoNobel erfolgen.

Presszeit

Unter Presszeit versteht man das Zeitintervall, während dessen die Leimfuge unter Pressdruck steht, bevor das Material weiterverarbeitet wird. Die Presszeit wird mittels genormter Analyseverfahren gemessen, so dass Presszeiten verschiedener Systeme miteinander verglichen werden können.

Zahlreiche Parameter beeinflussen die Leistungsfähigkeit eines Leimsystems, z. B. Zustand der Presse, Feuchtigkeitsgehalt der Fügeteile, Art des Bauteils und die Holzart.

Die vorgegebenen Presszeiten beziehen sich auf eine Materialtemperatur von 20°C. Wenn die Temperatur des Materials niedriger ist, muss die Presszeit verlängert werden. Materialtemperaturen < +18°C sind bei der Produktion von tragenden geklebten Holzbauteilen nach DIN 1052 nicht zulässig. Die in den Tabellen 1 + 2 angegebenen Werte dienen als Richtlinie.

Die Presszeiten werden nach DIN EN 302-6 bestimmt. Zur Brettschichtholz-Herstellung gemäß DIN 1052 werden normalerweise diese Presszeiten gewählt (s. Tabelle 2 unten).

Wenn durchgängig eine dünne Klebstofffuge (ca. 0,1 mm) gewährleistet ist, kann die Mindest-Presszeit niedriger sein als nach EN 302-6 festgelegt. Die Werte sind in Tabelle 1 aufgeführt (s. unten). In diesen Fällen muss die maximale Dicke der Leimfuge regelmäßig durch die firmeninterne Produktionskontrolle geprüft und die ordnungsgemäße Qualität der Leimfugen durch regelmäßige Delaminationsprüfungen nachgewiesen werden.

Tabelle 1: Presszeiten bei garantierter dünner Leimfuge (ca. 0,1mm)

Presszeiten bei garantierter dünner Leimfuge (250 g/m², ca. 0.1 mm)	**Leimfugentemperatur**	**Verhältnis 100:30**	**Verhältnis 100:100**
	20°C	3 h	1 h 30 min

Neben anderen Faktoren kann die Presszeit durch die Klebstofffugendicke beeinflusst werden. In Fällen, bei denen eine Klebstofffugendicke von ca. 0,1mm nicht garantiert werden kann, müssen die Presszeiten gemäß EN 302-6 eingehalten werden. Diese Mindest-Presszeit ist nachfolgend aufgelistet.

Tabelle 2: Presszeit gemäß EN 302-6

Presszeit gemäß EN302-6 (ca. 0.3 mm)	**Leimfugentemperatur**	**Verhältnis 100:30**	**Verhältnis 100:100**
	20°C	6 h 30 min	2 h 30 min

Die vorgegebenen Presszeiten beziehen sich auf die Produktion von geraden Bauteilen mit einer Holzfeuchte von ca. 12%. Bei Verleimung von gekrümmten Bauteilen oder Holz mit einem höheren Feuchtigkeitsgehalt muss die Presszeit verlängert werden.

Wenn die Brettschichtholz-Produktion bei erhöhten Temperaturen durchgeführt wird, entweder in einer Heißpresse oder bei Hochfrequenz-Aushärtung, kann die Presszeit verkürzt werden. In diesen speziellen

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Fällen muss stets ein AkzoNobel Anwendungstechniker hinzugezogen werden. Bevor Verleimungsbedingungen für eine spezielle Produktion festgelegt werden, müssen Delaminierungsprüfungen nach EN 14080:2013 Anhang C.4.3 oder C.4.4 durchgeführt und hierbei die Anforderungen gemäß EN14080:2013 Tabelle 9 erfüllt werden.

Pressdruck

Bei der Produktion von Brettschichtholz hängt der benötigte Pressdruck u. a. von der Stärke der Lamellen sowie der Holzart ab.

Lamellen mit einer Stärke unterhalb 35 mm erfordern einen Pressdruck zwischen 0.6 – 0.8 MPa. Lamellen mit einer Stärke zwischen 35 -45 mm benötigen einen Pressdruck von 0.8 MPa (genutete Lamellen) oder 1.0 MPa (nicht genutete Lamellen). Für Lamellen mit einer Stärke zwischen 45 – 80 mm sollte der Pressdruck bei 0.8 – 1.0 MPa liegen. Beachten Sie, dass Lamellen mit einer Stärke von mehr als 45 mm nicht zur Brettschichtholz-Produktion zugelassen sind. Bei getrenntem Auftrag von Leim und Härter kann derselbe Pressdruck für die Flächenverleimung verwendet werden.

Ein zu hoher Pressdruck verursacht einen zu hohen Leimaustritt, was zu einer schlechten Verklebung führt. Ein zu niedriger Pressdruck führt zu einem zu geringen Kontakt zwischen den zwei Oberflächen, wodurch die Qualität der Leimfuge beeinträchtigt wird.

Leimauftrag

Die Leimauftragsmenge kann, abhängig von Holzart, Holzfeuchte, relativer Luftfeuchtigkeit, Raumtemperatur, Press-Typ, Wartezeit und Hobelqualität, variieren. Die Leimauftragsuntergrenze sollte jedoch nicht niedriger sein als die Werte in nachfolgender Tabelle:

Die Leimauftragsmenge sollte bei Aushärtung bei Raumtemperatur nicht unter 220 g/m² liegen.
Die Leimauftragsmenge sollte bei Aushärtung mit Hochfrequenz nicht unter 180 g/m² liegen.

Bei der Herstellung von tragenden Bauteilen darf eine Reduzierung der Leimauftragsmenge, z.B. bei sehr kurzen Wartezeiten, nur unter Zustimmung der Anwendungstechnik unter Berücksichtigung der Produktionsparameter bei der jeweiligen Produktionslinie erfolgen. Diese Optimierung setzt voraus, dass die vorgegebenen Parameter eingehalten und kontinuierliche Kontrollen in Form von Delaminationsprüfungen durchgeführt werden. Eine schriftliche und signierte Bestätigung von AkzoNobel und der Klebstoffprüfstelle ist dafür zwingend erforderlich.

Ein geringes Herauspressen von Leim entlang der Leimfuge bei Anbringen des Pressdruckes weist sowohl auf einen angemessenen Leimauftrag als auch auf die Einhaltung der Wartezeit hin.
Starker Leimaustritt deutet auf einen zu hohen Leimauftrag, sehr hohen Pressdruck oder eine Kombination aus Beidem hin.
Wird eine längere Wartezeit erforderlich, kann ein höherer Leimauftrag gewählt werden.
Ein gleichmäßiger Leimauftrag ist sehr wichtig.

Holzfeuchte

Der Feuchtigkeitsgehalt des Holzes hat Auswirkungen auf das Verleimresultat. Eine hohe Holzfeuchte kann das System verlangsamen. Bei bestimmten Leimsystemen kann ein übermäßig hoher Feuchtigkeitsgehalt negative Auswirkungen auf die Leimfugenqualität haben.

In bestimmten Fällen kann eine viel zu geringe Holzfeuchte den Verklebungsprozess beschleunigen.

Der Feuchtigkeitsgehalt des Holzes hat auch eine Auswirkung auf die Gesamtqualität des Endproduktes. Eine ungleichmäßig, wesentlich zu hohe/niedrige Holzfeuchte kann zu Verzug, Schüsselung und

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Unebenheiten des Endproduktes führen.

Für die Brettschichtholz-Produktion sollte der Feuchtigkeitsgehalt vorzugsweise bei 10-12%, mindestens jedoch zwischen 8-15% liegen.

Vorbereitung des Holzes

Für beste Ergebnisse muss das Holz sauber gehobelt sein. Optimale Festigkeit wird erreicht, wenn die Verleimung spätestens 24 Stunden nach der Hobelung erfolgt.

Die Oberfläche muss frei von Staub, Fett, Öl und anderen Verunreinigungen sein.

Die Fügeteile müssen sorgfältig ausgewählt werden, um eine optimale Leimfugenqualität erzeugen zu können. Um die Presszeiten in der oben stehenden Tabelle einhalten zu können, muss die Lamellentemperatur mindestens 20°C betragen. Materialtemperaturen unterhalb +18°C sind zur Produktion von tragenden Bauteilen gemäß DIN1052 nicht zugelassen.

Das Klebstoffsystem ist für folgende Holzarten zugelassen: Fichte, Kiefer, Tanne, Europäische Lärche

Nachhärtung

Nach erfolgter Presszeit verfügt die Leimfuge der Konstruktion über genügend Festigkeit, um weiterverarbeitet zu werden. Die Endfestigkeit wird nach einer Zeit, die abhängig von der Presszeit/-temperatur sowie der Lagertemperatur ist, erreicht.

Unter Nachhärtezeit versteht man die Zeit die benötigt wird, damit die Leimfuge die vollständige Festigkeit und Wasserbeständigkeit erhält.
Die Nachhärtezeit hängt von der Presszeit, Presstemperatur, Lamellentemperatur sowie der Nachhärtetemperatur ab. Aushärtung bei anderen Temperaturen als 20°C verändert die benötigte Nachhärtezeit. Die erforderliche Nachhärtezeit muss von der Anwendungstechnik bestimmt werden.

Bei 20°C beträgt die Nachhärtezeit 40 Stunden bei 100:30 und 12 Stunden bei 100:100.

Zusätzliche Informationen zum Keilzinken
Für die Produktion von Keilzinkverbindungen müssen die Anforderungen wie in DIN 1052 und EN14080:2013 beschrieben, befolgt werden. Die Applikation muss im Untermischverfahren stattfinden. Die unten angeführte Tabelle beinhaltet wichtige Verarbeitungsparameter:

Nominales Mischungsverhältnis	Untermischverfahren (Fichte, Tanne, Kiefer): 100:10-100 Untermischverfahren (Europäische Lärche): 100:30-60
Empfohlene Leimauftragsmenge	zwischen 250 -350 g/m2
Maximale Wartezeit	5 min
Aushärtezeit	100:10 8 h 15 min 100:30 3 h 100:100 1 h 30 min

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Pressdruck	Gemäß EN 14080:2013

Gemischte Applikation bei Keilzinken
Im Untermischverfahren werden profilierte Auftragswalzen oder gleichwertige Applikationsgeräte empfohlen. Das Mischungsverhältniss beträgt von 100:10 (Leim:Härter) bis maximal 100:100 Gewichtsteile. Die Abweichung zwischen Leim und Härter darf maximal ±3 GWT betragen. Die Benetzung der Zinkenflanken mit dem Leim- Härtergemisch muss mindestens 75% betragen.
Die Topfzeit legt die mögliche Verarbeitungsdauer des Leimgemischs fest und muss deshalb gut überwacht werden. Die Tabelle unter „Topfzeit" beinhaltet Angaben der Verarbeitungsdauer für verschiedene Mischungsverhältnisse. Ein gekühltes Gemisch verlängert die Topfzeit, höhere Temperaturen verkürzen sie.

Aushärtung von Keilzinkverbindungen
Gemäß EN 14080:2013 beträgt die Mindestaushärtetemperatur +20°C. Bei Hochfrequenzaushärtung muss die Fugentemperatur mindestens 65°C betragen.

Weiterverarbeitung von keilgezinkten Lamellen
Eine direkte Weiterverarbeitung der Lamellen darf nur erfolgen, wenn Transport und Hobelung keinen mechanischen Einfluss auf die Keilzinkenverbindung ausüben. Andernfalls muss die in obiger Tabelle angegebene Aushärtezeit befolgt werden.

Endfestigkeit von Keilzinkverbindungen
Das Erreichen der Endfestigkeit ist von den Aushärtebedingungen und vom Klebstoffsystem abhängig. Wird Plus mit einem Mischungsverhältnis von 100:10 verarbeitet, so wird die volle Wasserfestigkeit bei 100:10 in 72 Stunden, bei 100:30 in 40 Stunden und bei 100:100 in 12 Stunden erreicht.

Qualitätskontrolle von Keilzinkverbindungen
Die Qualitätskontrolle muss gemäß der verwendeten Produktnorm erfolgen.

Handhabung und Umweltinformation

Reinigung
Maschine stets vor Aushärtung des Leimes mit lauwarmem Wasser reinigen! Ausgehärteter Leim muss manuell entfernt werden. Die Verwendung von Leimwaschmittel 4450 oder Reinigungsmittel 2704 erleichtert die Reinigung der Leimauftragsgeräte.

Reinigungsmittel 2704;

Zur Reinigung von Gießanlagen fügen Sie 50/50 (Gewichtsteile) der Mixtur aus Wasser und Reinigungsmittel 2704 in die Anlage. Pumpen Sie die Lösung ca. 4 Minuten im Kreislauf der Gießanlage und spülen Sie danach mit warmem Wasser.

Leimwaschmittel 4450;

Fügen Sie 1% Leimwaschmittel 4450 (in Relation zum Restleim in der Gießanlage) der Anlage zu. Lassen

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Sie die Gießanlage ca. 5 Minuten weiterlaufen, damit die Mixtur ausreichend vermischt wird. Danach kann die Anlage mit lauwarmem Wasser gewaschen werden.

Handhabung

Vermeiden Sie den direkten Kontakt mit Leim und Härter. Tragen Sie stets Handschuhe und Schutzbrille. Bei Hautkontakt reinigen Sie die betroffene Hautstelle umgehend mit Seife und lauwarmem Wasser. Aufgrund seines niedrigen pH-Wertes reagiert der Härter korrosiv auf Kupfer und kupferhaltige Legierungen. Es wird daher Stahl oder Plastik für den Einsatz im direkten Gebrauch mit dem Produkt empfohlen. Das Sicherheitsdatenblatt informiert Sie hinsichtlich Gesundheit und Sicherheit. Lesen Sie diese Informationen sorgfältig durch.

Mischbarkeit

Ob ein Produkt mit einem anderen Produkt mischbar ist (z. B. beim Wechsel von Leim oder Härter auf ein anderes Produkt) muss in jedem speziellen Fall ermittelt werden. Bitte sprechen Sie mit Ihrer Kontaktperson von AkzoNobel zwecks weiterer Informationen.

Abfallbehandlung

Leim – das Produkt Plus A011 ist nicht kennzeichnungspflichtig.

Härter - Abhängig von der Klassifizierung des Härters muss er als Sondermüll angesehen werden (siehe Sicherheitsdatenblatt, Abschnitt 13)

Leim-/Härtergemisch – Das ausgehärtete System gilt im Normalfalle nicht als Sondermüll.

Achtung! *Es können nationale und/oder regionale Unterschiede bei den Vorschriften vorherrschen. Bitte nehmen Sie Kontakt mit den für Sie zuständigen Behörden auf.*

Waschwasser-Behandlung

Chemische Ausfällung → kommunale Kläranlage mit biologischer Behandlung.

Die Zusätze 4410, 4411, 4412 und 4413 dienen der Verringerung von Leimrückständen im Leimwaschwasser. Diese Produkte agieren als Flockungsmittel, die die Leimpartikel konzentrieren und sedimentieren. Nach der Behandlung hat das Waschwasser einen geringeren Trockengehalt, wodurch einem Verstopfen von Rohren und Abflüssen vorgebeugt wird. Das entstandene Sediment kann, nachdem es ausgehärtet ist, als ungefährlicher Industrie-Müll entsorgt werden.

Auffangen von Leimwaschwasser

Leimwaschwasser kann auf einfache Art in leeren Leimfässern gesammelt werden. Abhängig von der Menge des anfallenden Leimwaschwassers sowie der Zeit, die für die Sedimentation nach der Ausfällung benötigt wird, sollten 2 oder mehr Leimfässer bereitgestellt werden.

Entsorgung von aufbereitetem Leimwaschwasser

Das aufbereitete Leimwaschwasser darf nicht ohne Zustimmung der lokalen Behörden in das Abwassernetz eingeleitet werden.

Entsorgung von Sediment

Wenn ein Fass mit Sediment gefüllt ist, lassen Sie es - möglichst bei hohen Temperaturen um die 50°C - stehen, bis die Ablagerungen ausgehärtet sind. Die Fässer mit den ausgehärteten Rückständen können später als ungefährlicher Industriemüll entsorgt werden. Bitte nehmen Sie Kontakt mit den für Sie zuständigen Behörden hinsichtlich einer fachgerechten Entsorgung auf.

Contact Information

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Version: 4 (2015-06-15)

Reason for changes: update according to EN14080:2013

Weitere Informationen finden Sie in den Produktinformationen für 4410/4411/4412/4413.

Mechanische Ausfällung → kommunale Kläranlage mit biologischer Behandlung Mechanische Ausfällung (Sedimentation) wird zur Reduzierung des Trockengehaltes in Waschwasser angewandt, um die Gefahr einer Verstopfung von Leitungen zu minimieren. Zur Ausfällung geeignete Behälter sind leere Fässer oder IBC, abhängig von der anfallenden Menge an Waschwasser. Der sich im Behälter befindliche Schlamm sollte getrocknet werden (vorzugsweise bei > 50°C) und kann später als ungefährlicher Industriemüll entsorgt werden. Der restliche Wasseranteil sollte nicht ohne ausdrückliche Genehmigung durch die regionalen Behörden in das Abwassernetz eingeleitet werden.

Achtung! Es können nationale und/oder regionale Unterschiede bei den behördlichen Bestimmungen vorherrschen. Bitte nehmen Sie Kontakt mit den für Sie zuständigen Behörden auf. Zu weiteren Fragen steht Ihnen der Sachverständige in Umweltfragen von AkzoNobel zur Verfügung.

Gesundheit und Sicherheit

Bitte beachten Sie das entsprechende Sicherheitsdatenblatt

Rechtliche Klausel:

Diese Information basiert auf Laborversuchen und praktischen Erfahrungen. Sie dient als Einführung vor dem Hintergrund, die für den Anwender bestmögliche Verarbeitungsmethode zu ermitteln. Da sich die Produktionsbedingungen des Anwenders außerhalb unseres Einflussbereiches befinden, übernehmen wir keine Verantwortung für die Verarbeitungsergebnisse, die von den jeweils vorherrschenden Bedingungen beeinflusst werden. Es werden in jedem Falle Durchführungen von Versuchsreihen sowie regelmäßige Überprüfungen empfohlen.

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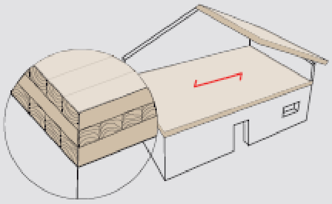
Version: 4 (2015-06-15)

Reason for changes: update according to EN14080:2013

C DESIGN TABLES

Design tables provided by manufacturers can be used as a starting point. The tables show possible configurations and indications based on span and loading.



Ceiling and roof structures

The structure of **L panels** is designed for use in ceiling and roof structures where the main loading is flexure. The outer layers are therefore oriented longitudinally to the panels.

Designation[1] [-]	Nominal thickness [mm]	Lamellar structure[2] [mm]	Dead load[3] [kN/m²]	Layers
L-60/3s	60	20 20 20	0.27	3
L-80/3s	80	30 20 30	0.36	3
L-90/3s	90	30 30 30	0.41	3
L-100/3s	100	40 20 40	0.45	3
L-110/3s	110	40 30 40	0.50	3
L-120/3s	120	40 40 40	0.54	3
L-130/5s	130	30 20 30 20 30	0.59	5
L-140/5s	140	40 20 20 20 40	0.63	5
L-150/5s	150	30 30 30 30 30	0.68	5
L-160/5s	160	40 20 40 20 40	0.72	5
L-170/5s	170	40 30 30 30 40	0.77	5
L-180/5s	180	40 30 40 30 40	0.81	5
L-200/5s	200	40 40 40 40 40	0.90	5
L-220/7s	220	40 20 40 20 40 20 40	0.99	7
L-240/7s	240	40 20 40 40 40 20 40	1.08	7
L-260/7s	260	40 30 40 40 40 30 40	1.17	7
L-280/7s	280	40 40 40 40 40 40 40	1.26	7
L-290/9s	290	40 30 30 30 30 30 30 30 40	1.31	9
L-310/9s	310	40 30 40 30 30 30 40 30 40	1.40	9
L-320/9s	320	40 30 40 30 40 30 40 30 40	1.44	9
L-360/9s	360	40 40 40 40 40 40 40 40 40	1.62	9
LL-190/7s	190	30 30 20 30 20 30 30	0.86	7
LL-210/7s	210	30 30 30 30 30 30 30	0.95	7
LL-230/7s	230	30 30 40 30 40 30 30	1.04	7
LL-240/7s	240	40 40 20 40 20 40 40	1.08	7
LL-260/7s	260	40 40 30 40 30 40 40	1.17	7
LL-280/7s	280	40 40 40 40 40 40 40	1.26	7
LL-300/9s	300	40 40 20 40 20 40 20 40 40	1.35	9
LL-330/9s	330	40 40 30 40 30 40 30 40 40	1.49	9
LL-360/9s	360	40 40 40 40 40 40 40 40 40	1.62	9
LL-400/11s	400	40 40 30 40 30 40 30 40 30 40 40	1.80	11

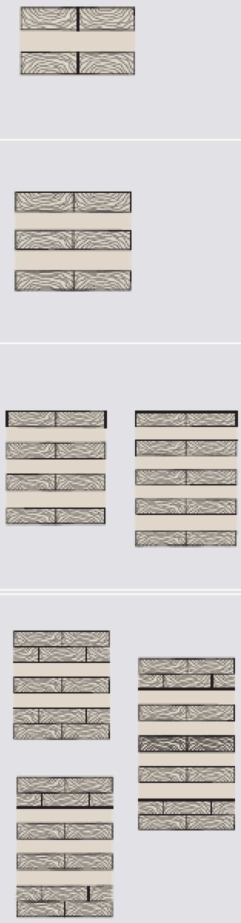
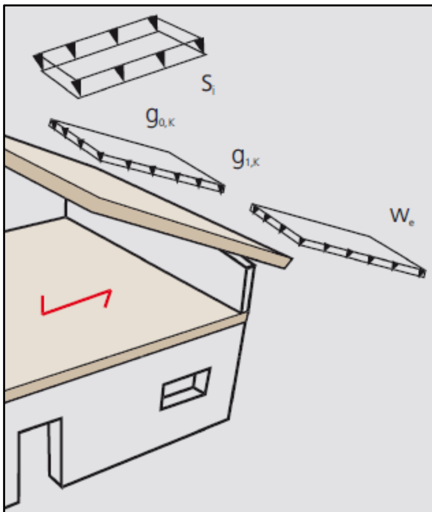


Figure 0.1: Possible configurations
Source: [10]

The tables can help to plan your projects, but they do not replace structural calculations. [10]



Application limits for cross-laminated timber components based on flexure[1] (F)

[kN/m²] Constant applied load $g_{1,k}$[2]	[kN/m²] Live load q_k[2]	Span length single-span beam L [m] 3,0	3,5	4,0	4,5	5,0	5,5	6,0	7,0
0.5	1.5	L-80/3s	L-90/3s	L-100/3s	L-110/3s	L-130/5s	L-140/5s	L-160/5s	LL-190/7s
	2.0				L-120/3s	L-140/5s	L-150/5s		
	3.0	L-90/3s	L-100/3s	L-110/3s	L-130/5s	L-150/5s	L-160/5s	L-180/5s	LL-210/7s
	4.0		L-110/3s	L-120/3s	L-140/5s	L-160/5s	L-180/5s	LL-190/7s	LL-230/7s
	5.0	L-100/3s		L-140/5s	L-160/5s	L-170/5s	LL-190/7s	LL-210/7s	LL-240/7s
1.0	1.5	L-80/3s	L-90/3s	L-110/3s	L-120/3s	L-140/5s	L-160/5s	L-170/5s	LL-190/7s
	2.0		L-100/3s		L-130/5s			L-180/5s	LL-210/7s
	3.0	L-90/3s		L-120/3s	L-140/5s	L-160/5s	L-170/5s	LL-190/7s	LL-230/7s
	4.0	L-100/3s	L-110/3s	L-130/5s	L-150/5s		LL-190/7s		LL-240/7s
	5.0		L-120/3s	L-140/5s	L-160/5s	L-180/5s		LL-210/7s	
1.5	1.5	L-90/3s	L-100/3s	L-110/3s	L-130/5s	L-150/5s	L-160/5s	L-180/5s	LL-210/7s
	2.0			L-120/3s	L-140/5s	L-160/5s	L-170/5s	LL-190/7s	LL-230/7s
	3.0	L-100/3s	L-110/3s	L-130/5s			L-180/5s		
	4.0		L-120/3s	L-140/5s	L-160/5s	L-170/5s	LL-190/7s	LL-210/7s	LL-240/7s
	5.0					L-180/5s			LL-260/7s
2.0	1.5	L-90/3s	L-100/3s	L-120/3s	L-140/5s	L-160/5s	L-170/5s	LL-190/7s	L-220/7s
	2.0		L-110/3s	L-130/5s			L-180/5s		LL-230/7s
	3.0	L-100/3s	L-120/3s	L-140/5s	L-160/5s	L-170/5s	LL-190/7s	LL-210/7s	LL-240/7s
	4.0					L-180/5s			
	5.0	L-110/3s	L-130/5s	L-150/5s	L-170/5s	L-190/7s	LL-210/7s	LL-230/7s	LL-260/7s

Table 10

Application limits for cross-laminated timber components based on vibration[4] (S)

[kN/m²] Constant applied load $g_{1,k}$[2]	[kN/m²] Live load q_k[2]	Span length single-span beam L [m] 3,0		3,5		4,0		4,5		5,0		5,5		6,0		7,0	
		S (≥6Hz)	S (≥8Hz)	S (≥6Hz)	S (≥8Hz)	S (≥6Hz)	S (≥8Hz)	S (≥6Hz)	S (≥8Hz)	S (≥6Hz)	S (≥8Hz)	S (≥6Hz)	S (≥8Hz)	S (≥6Hz)	S (≥8Hz)	S (≥6Hz)	S (≥8Hz)
0.5	1.5		L-80/3s	L-130/5s	L-90/3s	L-150/5s	L-100/3s	L-170/5s	L-110/3s	LL-190/7s	L-140/5s	LL-210/7s	L-160/5s	L-220/7s	L-190/7s	L-260/7s	LL-240/7s
	2.0								L-120/3s								
	3.0		L-90/3s		L-100/3s		L-110/3s		L-130/5s		L-150/5s						
	4.0				L-110/3s		L-120/3s		L-140/5s		L-160/5s		L-180/5s				
	5.0		L-100/3s				L-140/5s		L-160/5s		L-170/5s		LL-190/7s		LL-210/7s		
1.0	1.5		L-80/3s		L-90/3s		L-110/3s		L-130/5s		L-160/5s		L-180/5s				LL-260/7s
	2.0				L-100/3s												
	3.0		L-90/3s				L-120/5s		L-140/5s								
	4.0		L-100/3s		L-110/3s		L-130/5s		L-150/5s				LL-190/7s				
	5.0				L-120/3s		L-140/5s		L-160/5s								
1.5	1.5		L-90/3s		L-100/3s		L-120/3s		L-150/5s		L-180/5s						
	2.0														LL-230/7s		L-300/9s
	3.0		L-100/3s		L-110/3s		L-130/5s										
	4.0				L-120/3s		L-140/5s		L-160/5s								
	5.0																
2.0	1.5		L-90/3s		L-110/3s						LL-190/7s		LL-210/7s		LL-240/7s		
	2.0																
	3.0		L-100/3s		L-120/3s												
	4.0																
	5.0		L-110/3s		L-130/5s		L-150/5s		L-170/5s					LL-230/7s			

Figure 0.2: Design table - ceiling - single span
Source: [10]

D CALCULATIONS

CALCULATION EXAMPLES

MATERIAL PROPERTIES C24

Strength and stiffness properties in N/mm^2

Bending strength	fm;k	24
Tensile strength	ft;0;k	14
	ft;90;k	0,4
Compression strength	fc;0;k	21
	fc;90;k	2,5
Shear strength	fv;k	2,7
	fr;k	1
Modulus of elasticity	E0;mean	11000
	E90;mean	370
Shear modulus	Gmean	690
	Gr;mean	50
Density [kg/m^3]	ρk	350
	ρmean	420

Modification factor	kmod	0,8
Material factor	γM	1,3
Creep factor	kdef	0,9
Quasi-permanent	ψ2	0,3

$$k_l = \min\begin{cases} 1 + 0{,}025 \cdot n \\ 1{,}2 \end{cases}$$

Strength factor	kl	1,075

ASSUMPTIONS

The width of the elements is taken as 1m

CONTENTS

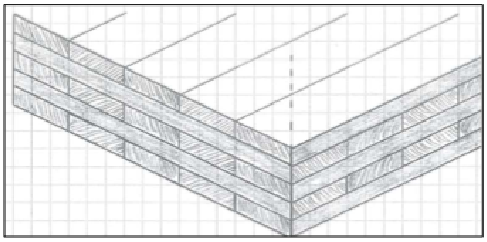
Example one: 5 layer CLT-element
Example two: 7 layer CLT-element
Example three: 7 layer HCCLT-element

EXAMPLE ONE: 5 LAYER CLT-ELEMENT

DIMENSIONS

Length, height and thicknesses in mm

Length span	l	7200
Thickness layer 1	h1	40
Thickness layer 2	h2	20
Thickness layer 3	h3	40
Thickness layer 4	h4	20
Thickness layer 5	h5	40
Total height	h	160



LOADING

Variable and permanent load in kN/m^2

Permanent		
- Self weight	ge	0,672
- Floor covering	g	1
Live load	q	2,8
(incl. separating walls)		
Design load	qd	6,46
Moment [kNm]	Mmax	41,84
Shear force [kN]	Vmax	23,25

COMPOSITE THEORY

Effective bending stiffness in Nmm^2

$$(EI)_{ef} = E_0 \cdot \frac{1}{12} \cdot b \cdot a_m^3 \cdot k_1$$

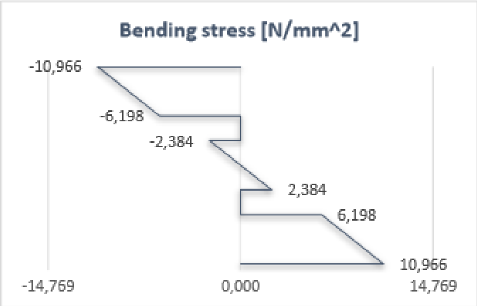
$$k_1 = 1 - (1 - \frac{E_{90}}{E_0}) \cdot \frac{(a_{m-2}^3 - a_{m-4}^3 + \cdots \pm a_1^3)}{a_m^3}$$

Composition factor	k1	0,894303977
	(EI)ef	3,35781E+12

Bending stress in N/mm^2

$$\sigma_m = \frac{M_{max}}{(EI)_{ef}} \cdot E_0 \cdot \frac{a_m}{2}$$

10,966 ≤ 14,769



MECHANICALLY JOINTED BEAMS THEORY

Effective bending stiffness

$$(EI)_{ef} = \sum (E_i \cdot I_i + E_i \cdot \gamma_i \cdot A_i \cdot a_i^2)$$

$$\gamma_1 = \frac{1}{1 + \frac{\pi^2 \cdot E_0 \cdot A_1 \cdot h_2}{G_r \cdot b \cdot l^2}}$$

Gamma factor γ1 0,96716

(EI)ef 3,23998E+12

Normall stresses, tension and compression in N/mm^2

$$\sigma_t = \frac{M_{max} \cdot E_0 \cdot \gamma_1 \cdot a_1}{(EI)_{ef}} \leq f_{t;d}$$

8,244 ≤ 8,615

$$\sigma_c = \frac{M_{max} \cdot E_0 \cdot \gamma_1 \cdot a_1}{(EI)_{ef}} \leq f_{c;d}$$

8,244 ≤ 12,923

Bending stress in N/mm^2

$$\sigma_{m,1} = \frac{M_{max} \cdot E_0 \cdot (\gamma_1 \cdot a_1 + 0{,}5 * h_1)}{(EI)_{ef}} \leq f_{m;d}$$

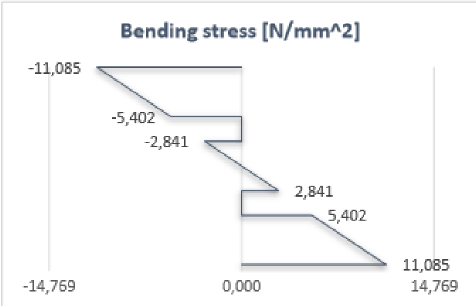
11,085 ≤ 14,769

$$\sigma_{m,2} = \frac{M_{max} \cdot E_0 \cdot (\gamma_1 \cdot a_1 - 0{,}5 * h_1)}{(EI)_{ef}} \leq f_{m;d}$$

5,402 ≤ 14,769

$$\sigma_{m,3} = \frac{M_{max} \cdot E_0 \cdot (0{,}5 * h_3)}{(EI)_{ef}} \leq f_{m;d}$$

2,841 ≤ 14,769



Shear stress in N/mm^2

$$\tau_{v;1} = \frac{V_{max} \cdot E_0 \cdot 0{,}5 \cdot A_1 \cdot (\gamma_1 \cdot a_1 + 0{,}25 \cdot h_1)}{(EI)_{ef} \cdot b} \leq f_{v;d}$$

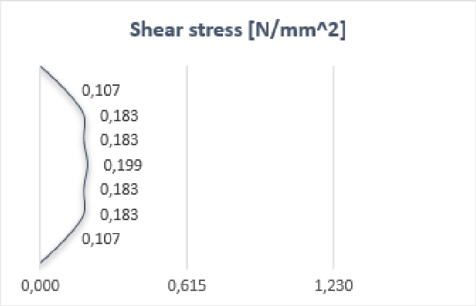
0,107 ≤ 1,662

$$\tau_{v;2} = \frac{V_{max} \cdot (E_0 \cdot A_1 \cdot \gamma_1 \cdot a_1)}{(EI)_{ef} \cdot b} \leq f_{v;d}$$

0,183 ≤ 1,662

$$\tau_{v;3} = \frac{V_{max} \cdot (E_0 \cdot A_1 \cdot \gamma_1 \cdot a_1 + E_0 \cdot 0{,}5 \cdot A_3 \cdot a_3)}{(EI)_{ef} \cdot b} \leq f_{v;d}$$

0,199 ≤ 1,662



$$\tau_r = \frac{V_{max} \cdot (E_0 \cdot A_1 \cdot \gamma_1 \cdot a_1)}{(EI)_{ef} \cdot b} \leq f_{r;d}$$

0,183 ≤ 0,615

SHEAR ANALOGY

Effective bending stiffness

$$(EI)_{ef} = \sum E_{x,i} \cdot d_i \cdot z_i^2 + \sum E_{x,i} \cdot \frac{d_i^3}{12}$$

(EI)ef 3,35781E+12

Bending stress in N/mm^2

$$\sigma_{m,1} = E_x \cdot \frac{M_{max}}{(EI)_{ef}} \cdot z$$

10,966 ≤ 14,769

$$\sigma_{m,2} = E_x \cdot \frac{M_{max}}{(EI)_{ef}} \cdot z$$

5,483 ≤ 14,769

$$\sigma_{m,3} = E_x \cdot \frac{M_{max}}{(EI)_{ef}} \cdot z$$

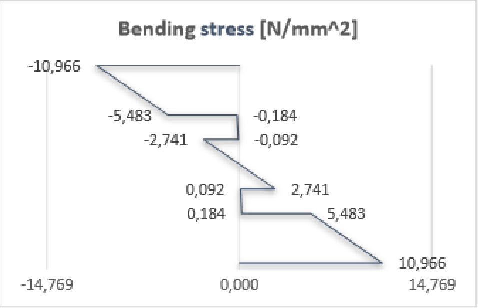
0,184 ≤ 14,769

$$\sigma_{m,4} = E_x \cdot \frac{M_{max}}{(EI)_{ef}} \cdot z$$

0,092 ≤ 14,769

$$\sigma_{m,5} = E_x \cdot \frac{M_{max}}{(EI)_{ef}} \cdot z$$

2,741 ≤ 14,769



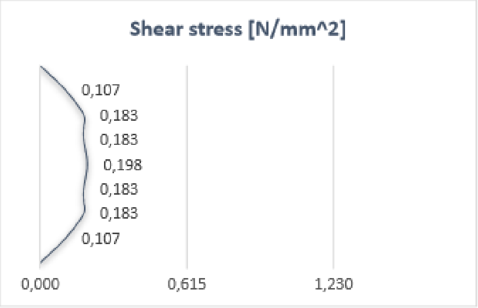
Shear stress in N/mm^2

$$\tau_{xz} = \frac{E \cdot S_x}{(EI)_{ef}} \cdot V_{max}$$

$$where:\ E \cdot S_x = \int_z^{d/2} E_x \cdot \bar{z} \cdot d\bar{z}$$

$$\tau_{xz,i/i+1} = \frac{E \cdot S_{x,i/i+1}}{(EI)_{ef}} \cdot V_{max}$$

$$where:\ E \cdot S_{x,i/i+1} = \sum_{j=i+1}^{n} E_{x,j} \cdot z_j \cdot d_j$$



DEFLECTION

Instantaneous and final deflection in mm

$$w = \frac{5}{384} \cdot \frac{q \cdot l^4}{EI}$$

$$w_{fin,G} = w_{inst,G} \cdot (1 + k_{def})$$
$$w_{fin,Q} = w_{inst,Q} \cdot (1 + \psi_2 \cdot k_{def})$$

Instantaneous deflection

winst,G	18,058	$w_{inst} \leq l/300$			
winst,Q	30,240		48,298	≤	24,000
winst	48,298				

Final deflection

wfin,G	34,310	$w_{fin} \leq l/250$			
wfin,Q	38,405		72,715	≤	28,800
wfin	72,715				

Including shear deformation

$$w = \frac{5}{384} \cdot \frac{q \cdot l^4}{EI} + \frac{q \cdot l^2}{8 \cdot GA}$$

$$\frac{q \cdot l^2}{8 \cdot GA}, with\ GA = \frac{a^2}{\frac{h_1}{2 \cdot G_1 \cdot b} + \sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b} + \frac{h_n}{2 \cdot G_n \cdot b}}$$

winst 50

VIBRATION

Frequency in Hz

$$f_1 = \frac{\pi}{2 \cdot l^2} \cdot \sqrt{\frac{EI}{m}}$$

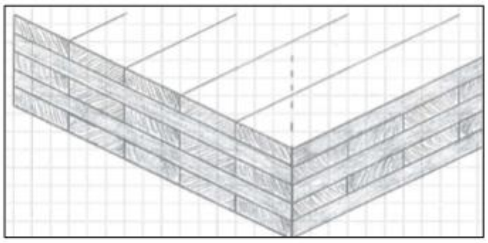
Fundamental frequency f1 4,218

EXAMPLE TWO: 7 LAYER CLT-ELEMENT

DIMENSIONS

Length, height and thicknesses in mm

Length span	l	7200
Thickness layer 1	h1	40
Thickness layer 2	h2	20
Thickness layer 3	h3	40
Thickness layer 4	h4	40
Thickness layer 5	h5	40
Thickness layer 6	h6	20
Thickness layer 7	h7	40
Total height	h	240



LOADING

Variable and permanent load in kN/m^2

Permanent		
- Self weight	ge	1,008
- Floor covering	g	1
Live load (incl. separating walls)	q	2,8
Design load	qd	6,91
Moment [kNm]	Mmax	44,78
Shear force [kN]	Vmax	24,88

COMPOSITE THEORY

Effective bending stiffness in Nmm^2

$$(EI)_{ef} = E_0 \cdot \frac{1}{12} \cdot b \cdot a_m^3 \cdot k_1$$

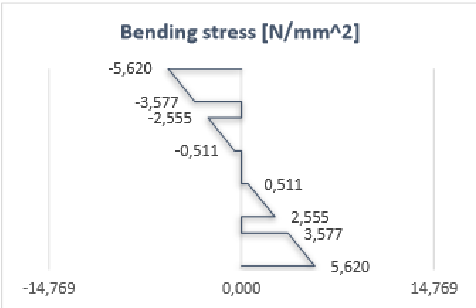
$$k_1 = 1 - (1 - \frac{E_{90}}{E_0}) \cdot \frac{(a_{m-2}^3 - a_{m-4}^3 + \cdots \pm a_1^3)}{a_m^3}$$

Composition factor	k1	0,82999
	(EI)ef	1,05177E+13

Bending stress in N/mm^2

$$\sigma_m = \frac{M_{max}}{(EI)_{ef}} \cdot E_0 \cdot \frac{a_m}{2}$$

5,620 $\leq$ 14,769



MECHANICALLY JOINTED BEAMS THEORY

Effective bending stiffness

$$(EI)_{ef} = \sum (E_i \cdot I_i + E_i \cdot \gamma_i \cdot A_i \cdot a_i^2) \qquad \gamma_i = \frac{1}{1 + \frac{\pi^2 \cdot E_0 \cdot A_i \cdot h_{cross}}{G_r \cdot b \cdot l^2}}$$

Gamma factors

- inner layer	γ1	0,96716
- outer layer	γ2	0,96716
	(EI)ef	1,01075E+13

Normall stresses, tension and compression in N/mm^2

$$\sigma_t = \frac{M_{max} \cdot E_0 \cdot (\gamma_1 \cdot a_1 + \gamma_2 \cdot (a_2 - a_1))}{(EI)_{ef}} \leq f_{t;d} \qquad \sigma_c = \frac{M_{max} \cdot E_0 \cdot (\gamma_1 \cdot a_1 + \gamma_2 \cdot (a_2 - a_1))}{(EI)_{ef}} \leq f_{c;d}$$

4,714 ≤ 8,615   4,714 ≤ 12,923

Bending stress in N/mm^2

$$\sigma_{m,1} = \frac{M_{max} \cdot E_0 \cdot (\gamma_1 \cdot a_1 + \gamma_2 \cdot (a_2 - a_1) + 0{,}5 * h_1)}{(EI)_{ef}} \leq f_{m;d}$$

5,688 ≤ 14,769

$$\sigma_{m,2} = \frac{M_{max} \cdot E_0 \cdot (\gamma_1 \cdot a_1 + \gamma_2 \cdot (a_2 - a_1) - 0{,}5 * h_1)}{(EI)_{ef}} \leq f_{m;d}$$

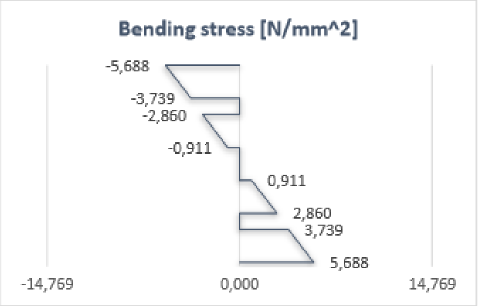
3,739 ≤ 14,769

$$\sigma_{m,3} = \frac{M_{max} \cdot E_0 \cdot (\gamma_1 \cdot a_1 + 0{,}5 * h_3)}{(EI)_{ef}} \leq f_{m;d}$$

2,860 ≤ 14,769

$$\sigma_{m,4} = \frac{M_{max} \cdot E_0 \cdot (\gamma_1 \cdot a_1 - 0{,}5 * h_3)}{(EI)_{ef}} \leq f_{m;d}$$

0,911 ≤ 14,769



Shear stress in N/mm^2

$$\tau_{v;1} = \frac{V_{max} \cdot (E_0 \cdot 0{,}5 \cdot A_1 \cdot (\gamma_1 \cdot a_1 + \gamma_2 \cdot (a_2 - a_1) + 0{,}25 \cdot h_1))}{(EI)_{ef} \cdot b} \leq f_{v;d}$$

0,058 ≤ 1,662

$$\tau_{v;2} = \frac{V_{max} \cdot (E_0 \cdot A_1 \cdot (\gamma_1 \cdot a_1 + \gamma_2 \cdot (a_2 - a_1)))}{(EI)_{ef} \cdot b} \leq f_{v;d}$$

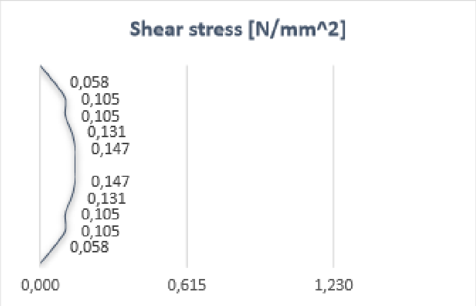
0,105 ≤ 1,662

$$\tau_{v;3} = \frac{V_{max} \cdot (E_0 \cdot A_1 \cdot (\gamma_1 \cdot a_1 + \gamma_2 \cdot (a_2 - a_1)) + E_0 \cdot 0{,}5 \cdot A_3 \cdot (\gamma_1 \cdot a_1 + 0{,}25 \cdot h_3))}{(EI)_{ef} \cdot b} \leq f_{v;d}$$

0,131 ≤ 1,662

$$\tau_{v;4} = \frac{V_{max} \cdot (E_0 \cdot A_1 \cdot (\gamma_1 \cdot a_1 + \gamma_2 \cdot (a_2 - a_1)) + E_0 \cdot A_3 \cdot \gamma_1 \cdot a_1)}{(EI)_{ef} \cdot b} \leq f_{v;d}$$

0,147 ≤ 1,662



$$\tau_{r;1} = \frac{V_{max} \cdot (E_0 \cdot A_1 \cdot (\gamma_1 \cdot a_1 + \gamma_2 \cdot (a_2 - a_1)))}{(EI)_{ef} \cdot b} \leq f_{r;d}$$

0,105 ≤ 0,615

$$\tau_{r;2} = \frac{V_{max} \cdot (E_0 \cdot A_1 \cdot (\gamma_1 \cdot a_1 + \gamma_2 \cdot (a_2 - a_1)) + E_0 \cdot A_3 \cdot \gamma_1 \cdot a_1)}{(EI)_{ef} \cdot b} \leq f_{r;d}$$

0,147 ≤ 0,615

SHEAR ANALOGY

Effective bending stiffness

$$(EI)_{ef} = \sum E_{x,i} \cdot d_i \cdot z_i^2 + \sum E_{x,i} \cdot \frac{d_i^3}{12}$$

(EI)ef 1,04427E+13

Bending stress in N/mm^2

$$\sigma_{m,1} = E_x \cdot \frac{M_{max}}{(EI)_{ef}} \cdot z$$

5,661 ≤ 14,769

$$\sigma_{m,2} = E_x \cdot \frac{M_{max}}{(EI)_{ef}} \cdot z$$

3,774 ≤ 14,769

$$\sigma_{m,3} = E_x \cdot \frac{M_{max}}{(EI)_{ef}} \cdot z$$

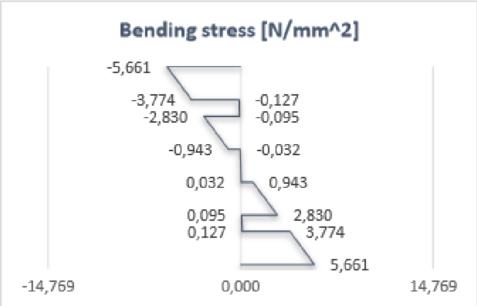
0,127 ≤ 14,769

$$\sigma_{m,4} = E_x \cdot \frac{M_{max}}{(EI)_{ef}} \cdot z$$

0,095 ≤ 14,769

$$\sigma_{m,5} = E_x \cdot \frac{M_{max}}{(EI)_{ef}} \cdot z$$

2,830 ≤ 14,769



$$\sigma_{m,6} = E_x \cdot \frac{M_{max}}{(EI)_{ef}} \cdot z$$

0,943 ≤ 14,769

$$\sigma_{m,7} = E_x \cdot \frac{M_{max}}{(EI)_{ef}} \cdot z$$

0,032 ≤ 14,769

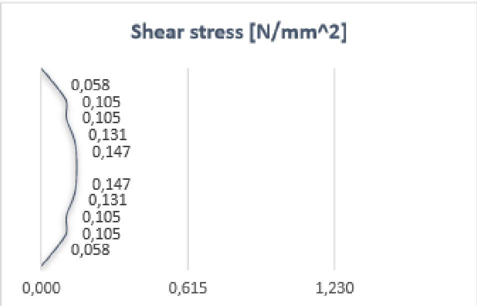
Shear stress in N/mm^2

$$\tau_{xz} = \frac{E \cdot S_x}{(EI)_{ef}} \cdot V_{max}$$

$$where:\ E \cdot S_x = \int_z^{d/2} E_x \cdot \bar{z} \cdot d\bar{z}$$

$$\tau_{xz,i/i+1} = \frac{E \cdot S_{x,i/i+1}}{(EI)_{ef}} \cdot V_{max}$$

$$where:\ E \cdot S_{x,i/i+1} = \sum_{j=i+1}^{n} E_{x,j} \cdot z_j \cdot d_j$$



DEFLECTION

Instantaneous and final deflection in mm

$$w = \frac{5}{384} \cdot \frac{q \cdot l^4}{EI}$$

$$w_{fin,G} = w_{inst,G} \cdot (1 + k_{def})$$
$$w_{fin,Q} = w_{inst,Q} \cdot (1 + \psi_2 \cdot k_{def})$$

Instantaneous deflection

winst,G	6,952	$w_{inst} \leq l/300$			
winst,Q	9,694		16,645	≤	24,000
winst	16,645				

Final deflection

wfin,G	13,208	$w_{fin} \leq l/250$			
wfin,Q	12,311		25,519	≤	28,800
wfin	25,519				

Including shear deformation

$$w = \frac{5}{384} \cdot \frac{q \cdot l^4}{EI} + \frac{q \cdot l^2}{8 \cdot GA}$$

$$\frac{q \cdot l^2}{8 \cdot GA}, with\ GA = \frac{a^2}{\frac{h_1}{2 \cdot G_1 \cdot b} + \sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b} + \frac{h_n}{2 \cdot G_n \cdot b}}$$

winst 18

VIBRATION

Frequency in Hz

$$f_1 = \frac{\pi}{2 \cdot l^2} \cdot \sqrt{\frac{EI}{m}}$$

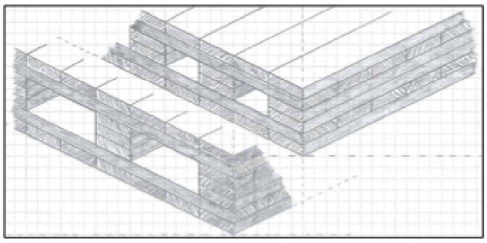
Fundamental frequency f1 6,798

EXAMPLE THREE: 7 LAYER HCCLT-ELEMENT

DIMENSIONS

Length, height and thicknesses in mm

Length span	l	7200
Width of a single board	b1	200
Thickness layer 1	h1	40
Thickness layer 2	h2	20
Thickness layer 3	h3	40
Thickness layer 4	h4	40
Thickness layer 5	h5	40
Thickness layer 6	h6	20
Thickness layer 7	h7	40
Total height	h	240



LOADING

Variable and permanent load in kN/m^2

Permanent		
- Self weight	ge	0,672
- Floor covering	g	1
Live load	q	2,8
(incl. separating walls)		
Design load	qd	6,46
Moment [kNm]	Mmax	41,84
Shear force [kN]	Vmax	23,25

COMPOSITE THEORY

Effective bending stiffness in Nmm^2

$$(EI)_{ef} = E_0 \cdot \frac{1}{12} \cdot b \cdot a_m^3 \cdot k_1$$

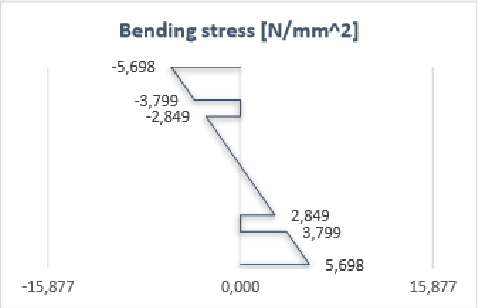
$$k_1 = 1 - (1 - \frac{E_{90}}{E_0}) \cdot \frac{(a_{m-2}^3 - a_{m-4}^3 + \cdots \pm a_1^3)}{a_m^3}$$

Composition factor	k1	0,83447
	(EI)ef	9,69315E+12

Bending stress in N/mm^2

$$\sigma_m = \frac{M_{max}}{(EI)_{ef}} \cdot E_0 \cdot \frac{a_m}{2}$$

5,698 ≤ 15,877



MECHANICALLY JOINTED BEAMS THEORY

Effective bending stiffness

$$(EI)_{ef} = \sum (E_i \cdot I_i + E_i \cdot \gamma_i \cdot A_i \cdot a_i^2) \qquad \gamma_i = \frac{1}{1 + \frac{\pi^2 \cdot E_0 \cdot A_i \cdot h_{cross}}{G_r \cdot b \cdot l^2}}$$

Gamma factor	γ1	0,90756
	(EI)ef	8,63189E+12

Normall stresses, tension and compression in N/mm^2

$$\sigma_t = \frac{M_{max} \cdot E_0 \cdot \gamma_1 \cdot a_1}{(EI)_{ef}} \leq f_{t;d} \qquad \sigma_c = \frac{M_{max} \cdot E_0 \cdot \gamma_1 \cdot a_1}{(EI)_{ef}} \leq f_{c;d}$$

4,839 ≤ 8,615   4,839 ≤ 12,923

Bending stress in N/mm^2

$$\sigma_{m,1} = \frac{M_{max} \cdot E_0 \cdot (\gamma_1 \cdot a_1 + 0{,}5 * h_1)}{(EI)_{ef}} \leq f_{m;d}$$

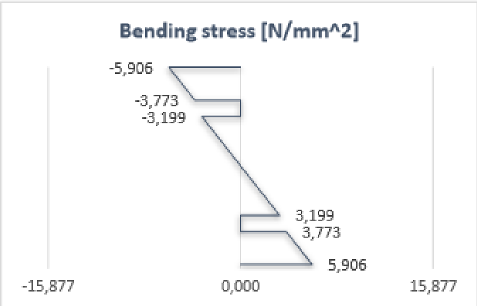
5,906 ≤ 15,877

$$\sigma_{m,2} = \frac{M_{max} \cdot E_0 \cdot (\gamma_1 \cdot a_1 - 0{,}5 * h_1)}{(EI)_{ef}} \leq f_{m;d}$$

3,773 ≤ 15,877

$$\sigma_{m,3} = \frac{M_{max} \cdot E_0 \cdot (0{,}5 * h_4 + h_3)}{(EI)_{ef}} \leq f_{m;d}$$

3,199 ≤ 15,877



Shear stress in N/mm^2

$$\tau_{v;1} = \frac{V_{max} \cdot (E_0 \cdot 0{,}5 \cdot A_1 \cdot (\gamma_1 \cdot a_1 + 0{,}25 \cdot h_1))}{(EI)_{ef} \cdot b} \leq f_{v;d}$$

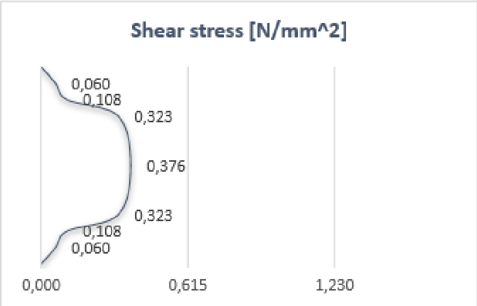
0,060 ≤ 1,662

$$\tau_{v;2} = \frac{V_{max} \cdot (E_0 \cdot A_1 \cdot \gamma_1 \cdot a_1)}{(EI)_{ef} \cdot b} \leq f_{v;d}$$

0,108 ≤ 1,662

$$\tau_{v;3} = \frac{V_{max} \cdot (E_0 \cdot A_1 \cdot \gamma_1 \cdot a_1 + E_0 \cdot 0{,}5 \cdot A_3 \cdot a_3)}{(EI)_{ef} \cdot b} \leq f_{v;d}$$

0,376 ≤ 1,662



$$\tau_r = \frac{V_{max} \cdot (E_0 \cdot A_1 \cdot \gamma_1 \cdot a_1)}{(EI)_{ef} \cdot b} \leq f_{r;d}$$

0,323 ≤ 0,615

SHEAR ANALOGY

Effective bending stiffness

$$(EI)_{ef} = \sum E_{x,i} \cdot d_i \cdot z_i^2 + \sum E_{x,i} \cdot \frac{d_i^3}{12}$$

(EI)ef 9,44533E+12

Bending stress in N/mm^2

$$\sigma_{m,1} = E_x \cdot \frac{M_{max}}{(EI)_{ef}} \cdot z$$

5,848 ≤ 15,877

$$\sigma_{m,2} = E_x \cdot \frac{M_{max}}{(EI)_{ef}} \cdot z$$

3,898 ≤ 15,877

$$\sigma_{m,3} = E_x \cdot \frac{M_{max}}{(EI)_{ef}} \cdot z$$

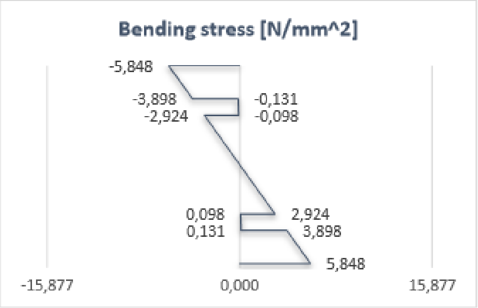
0,131 ≤ 15,877

$$\sigma_{m,4} = E_x \cdot \frac{M_{max}}{(EI)_{ef}} \cdot z$$

0,098 ≤ 15,877

$$\sigma_{m,5} = E_x \cdot \frac{M_{max}}{(EI)_{ef}} \cdot z$$

2,924 ≤ 15,877



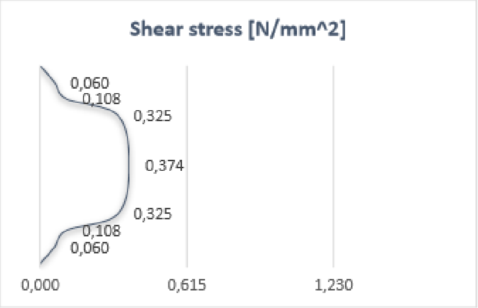
Shear stress in N/mm^2

$$\tau_{xz} = \frac{E \cdot S_x}{(EI)_{ef}} \cdot V_{max}$$

$$where:\ E \cdot S_x = \int_z^{d/2} E_x \cdot \bar{z} \cdot d\bar{z}$$

$$\tau_{xz,i/i+1} = \frac{E \cdot S_{x,i/i+1}}{(EI)_{ef}} \cdot V_{max}$$

$$where:\ E \cdot S_{x,i/i+1} = \sum_{j=i+1}^{n} E_{x,j} \cdot z_j \cdot d_j$$



DEFLECTION

Instantaneous and final deflection in mm

$$w = \frac{5}{384} \cdot \frac{q \cdot l^4}{EI}$$

$$w_{fin,G} = w_{inst,G} \cdot (1 + k_{def})$$
$$w_{fin,Q} = w_{inst,Q} \cdot (1 + \psi_2 \cdot k_{def})$$

Instantaneous deflection

winst,G	6,778	$w_{inst} \leq l/300$			
winst,Q	11,351		18,129	≤	24,000
winst	18,129				

Final deflection

wfin,G	12,878	$w_{fin} \leq l/250$			
wfin,Q	14,415		27,293	≤	28,800
wfin	27,293				

Including shear deformation

$$w = \frac{5}{384} \cdot \frac{q \cdot l^4}{EI} + \frac{q \cdot l^2}{8 \cdot GA}$$

$$\frac{q \cdot l^2}{8 \cdot GA}, with\ GA = \frac{a^2}{\frac{h_1}{2 \cdot G_1 \cdot b} + \sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b} + \frac{h_n}{2 \cdot G_n \cdot b}}$$

winst 19

VIBRATION

Frequency in Hz

$$f_1 = \frac{\pi}{2 \cdot l^2} \cdot \sqrt{\frac{EI}{m}}$$

Fundamental frequency f1 6,885

CROSS LAYER CHECK

Additional checks are performed for the cross layer over the hollow cores

Lenght span	lc	600

Loading

Permanent		
- Self weight	ge	0,25
- Floor covering	g	1,00
Live load	q	2,50
(incl. separating walls)		
Design load	qd	5,44
Moment [kNm]	Mmax	0,24
Shear force [kN]	Vmax	1,63
Section modulus	W	66666,67

Bending

$$\frac{M}{W} \leq f_{m,d}$$

3,672 ≤ 15,877

Shear

$$\frac{1{,}5 \cdot V}{A} \leq f_{v,d}$$

0,122 ≤ 1,662

In longitudinal direction, only the cross section of the transverse layer is loaded.

$$\tau = \frac{V_{max} \cdot (E_0 \cdot A_{long\ span} \cdot \gamma_1 \cdot a_1)}{(EI)_{ef} \cdot h_{cross\ layer}} \leq f_{v,d}$$

1,075 ≤ 1,662